AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

704.

(Vol. XXXI.—May, 1894.)

DREDGING OPERATIONS.—DISCUSSION ON PAPER No. 689.*

By ROBERT A. CUMMINGS, FOSTER CROWELL, J. D. VAN BUREN, E. B. GOSLING, F. A. WASHBURN, R. S. BUCK, R. L. HARRIS, CHARLES B. BRUSH and PETER C. HAINS.

ROBERT A. CUMMINGS, Assoc. M. Am. Soc. C. E. (by letter).—In all river and harbor improvements the dredging operations are perhaps the most important, and the appliances best adapted to the varying conditions should be very carefully considered. The work greatly depends upon the dredging operations ; and, in this instance, the appliances used seem to indicate a very elaborate and interesting plant. It is, however, to be regretted that the author has not conveyed to us more information as to the dredgers and their mechanical efficiency, together with other details of operation. The length of time this work was in hand would make a detailed statement of the cost of the operations very valuable. Assuming the material was fine sand and mud, the location was very favorable for hydraulic dredgers.

In hydraulic dredgers, the height of the lift of the material is reduced to a minimum, and by using telescopic suction pipes the

* "Reclamation of the Potomac Flats at Washington, D. C.," by Peter C. Hains, M. Am. Soc. C. E.

dredgers would be independent of the variable height of the tide. The disadvantage of hydraulic dredgers, however, is that, in addition to the material lifted, they have to pump a greater percentage of water, which simply means a waste of power.

The author states that where "30 to 40% of the volume pumped is solid, such large quantities do not give good results"; this is undoubtedly the case where centrifugal pumps are used in combination with "agitators" or "mixers"; but does it not depend upon the class of material? It would not be hazardous to venture the opinion that vacuum pumps will handle the material in greater percentages and without being mixed by "agitators" or "mixers." The writer has designed such a hydraulic dredging appliance, and hopes to present, at no distant date, the result of experiments with the same. With centrifugal pumps, it has been found for silt and alluvial deposits 30 to 40% is the best proportion, and for fine gravel and coarse sand 10% of the volume pumped. The settling qualities of the material determine the best percentage to the volume pumped, and the pumping power should be proportioned accordingly. The percentage of solids pumped is obtained by taking samples from the discharge pipes and measuring the water and solids deposited in a graduated measure.

Another inherent defect of hydraulic dredgers is the impossibility of the captain knowing what the pump is doing; he has no positive knowledge whether he is pumping only water or a large percentage of solids; this feature is very important. The writer's improvements, referred to above, excavate a graded bottom by the self-feeding suction pipe. This is an advantage over ordinary hydraulic dredgers, which always leave the bottom full of very large pitholes and uneven places.

The author says: "The sand settled and packed in the discharge pipe, reducing its capacity." It would seem from this that the necessity of determining the settling qualities of the material in flowing water is necessary. The exceptional length of 3 000 ft. of the discharge pipe is, no doubt, accountable for a great part of the trouble. If the pump had been increased in speed or a relay pump placed about midway in the discharge pipe, the velocity of the water would have been greater, and the opportunity for settlement reduced to a minimum. It is not ordinarily economical to use the generally rough discharge pipes more than 1 500 ft. long.

Discharge pipes on pontoons are allowed to float about uncontrolled, having a number of slack lengths of pipe, to permit the movement of the dredger. These slack lengths of pipe often form sharp right angles to one another, owing to the unnecessary flexibility of the rubber joints. The waves from passing vessels or from wind raise one pontoon and its length of pipe above the next pontoon and its length of pipe. This strains the joint and often wrenches the fastenings so as to produce a bad leak, which causes expensive repairs and delays. The loss of head in discharge pipes due to eddies from swollen and uneven joints, joint-knees and excessive friction, are causes of much loss of power. An iron or wood discharge pipe floating on the surface of the water is a better arrangement. This can be accomplished by means of pairs of buoys connected by sling chains under the pipe and placed where necessary.

The author has referred to the velocity of the water flowing through the discharge pipe; it is hoped the method of determining the velocity of the water under pressure will be explained.

A statement of cost, with other details about the plant, interest, depreciation, repairs, coal, stores, etc., would be of much value.

In order to determine the mechanical work done, it is necessary to know the weight of the material and other conditions which may be given with advantage.

The author is to be congratulated on his comprehensive and instructive paper.

Foster Crowell, M. Am. Soc. C. E.—I should like to inquire more particularly about the prices. As stated in the abstract, the price for material handled twice by dredges and once in cars, and leveled by water-jet, was 21.2 cents, scow measurement, plus, I understand, 2 cents, the cost of trestle divided by the yardage; but there is a palpable error in the latter item. It should be 5.4 cents, if the figures of total cost of trestle and quantity of material moved over it are correctly given, making the total cost 26.6 cents; this, as I understand, is scow measurement.

Against it we have the results of the hydraulic dredges, given at about 14 cents, place measurement, with either pump used, which, on the basis of 20%, would correspond to about 11.5 cents, scow measurement, or considerably less than half, instead of a little more than half, as stated; that is, the difference appears to have been about 15 cents

a yard in favor of the hydraulic work, or, omitting the cost of trestle, about 9.75 cents per cubic yard saved by the hydraulic process. It is interesting to note that this saving would appear to be fairly represented by the second handling of the material by the plain dredge, but, as the combined lifts of the plain dredges were 60 ft., while the hydraulic dredge lifted only 27 ft., the economy of movement would be with the former; in other words, if the material had been deposited by the hydraulic dredges at a height within the reach of the plain dredges, the economy would have been on the side of the latter. This has a very important bearing on purely dredging operations where the material is to be moved afloat, and it would appear from these figures that, for scow work, with the character of material encountered on the Potomac Flats, the plain dredge would give the better results. Col. Hains' opinion on this point would be valuable.

JOHN D. VAN BUREN, M. Am. Soc. C. E.—It may be of interest, in connection with the description of the dredging operations contained in the paper, to give the results of a successful operation recently completed in the city of Newburgh. Without attempting any formal description, I will describe as briefly as possible what we have done.

We have a lake, which constitutes our reservoir, of about 200 acres, and by the raising of the lake it was made to overflow extensive flats, aggregating about 50 acres.

The depth of the water before dredging varied from nothing to about 9 ft., and the depth of muck from 1 to 12 ft., with an average of about 6 ft. The result was that the water was very much injured, becoming almost unfit for use. It was decided to clean out these flats, and, in order to do so, without injuring or wasting the water during the operation, it was necessary to be very careful in choosing the method. Centrifugal pumps were excluded because they would require a stirrer to feed the suction, thus soiling the water, and wasting too much. The method finally approved by the engineers and adopted was proposed by Messrs. Babcock, Lary & Co., contractors of Newburgh, N. Y., who skillfully and successfully completed the work. The method adopted was this. A large scow was built, with a bin in the bow, about 10 by 20 ft., and about 4 ft. deep. A vacuum pump or chamber was placed upon the scow and fed by two boilers. The suction pipe led from the bin to the vacuum chamber with an intervening inlet gate. A discharge pipe led through the

scow, with a discharge gate at the chamber. A derrick, with a swinging boom, supporting a Heywood bucket, holding 2 yds. of material, was mounted on the bow with the necessary hoisting machinery. This discharge pipe was connected with a 15-in. pipe placed upon pontoons made of barrels and carried to the shore, where it was connected with the shore line leading to the dumps. We carried it to a distance of from 500 to 3 000 ft., with a maximum lift of about 44 ft.

The operation was this. The bucket lifted the material into the bin, the vacuum was formed in the usual manner, the inlet gate was lifted, and the charge filled the chamber. Then the inlet gate was shut, the outlet gate opened, steam was let in, and the muck was driven to the dumps. The material was almost wholly muck, varying from a consistency of peat to light, fibrous material.

In order to protect the water as much as possible during the operation, we put canvas curtains across the bays, and by that method we intercepted all circulation with the main body of water. By this method we removed about 40C 000 yds.

Comparison has been made in the paper between the scow measurement and measurement in place. The result in this case was, the bucket measurement, which we tallied, was some 15 to 17% less than the measurement by cross-section; that is, the muck occupied more space in the bottom of the lake than it did in the bucket. We found this dredge was not so well calculated to remove stiff clay; probably it would remove sand and material of that kind. As to the additional quantity of water used, we found we could reduce it to about 15 per cent; that is, the quantity of solid material discharged was about 85% of the total discharge passing the pipe.

The operation was a complete success, and we carried it through without injuring the water in the slightest degree. The water has been good since the completion of the work of last year—which formed the greater part of the excavation.

Mr. Crowell.—What was the cost?

Mr. Van Buren.—It was done by contract, and the bid was 28 cents; we had bids by prominent firms ranging as high as 50 cents; the lowest was 28.

H. W. Brinckerhoff, M. Am. Soc. C. E.—What was the pressure?

Mr. Van Buren.—The pressure was about 50 lbs. We had some trouble from the stones occasionally lifted, or old trunks of trees, or

something of that kind, but not much. There was considerable trouble at first with the flexible india-rubber joints of the pipe line, but by having them made with spiral-wire wrapping, the trouble ceased.

Edgar B. Gosling, Jun. Am. Soc. C. E.—I have seen dredges discharging through pipes on pontoons in which the difficulty of the breaking of the joints, just mentioned, was overcome, to a great extent, by having the pontoons connected by chains, which did not allow the angle made by the pipes to change but a small number of degrees and thereby not strain the joints as much as where such chains were not used. That kind of discharge was used where the material had to be spread over large surfaces. At the same time there was dredging going on close by (this was, I may say, at the Suez Canal) where the endless-chain dredge, with a floating structure with a large discharge pipe or chute carried by means of a cantilever, was used. I have seen these cantilevers, from 200 to 300 ft. long, discharging the dredged material on the sides of the canal quite a distance in shore, so as not to bring undue pressure on the banks. It was considered economical to use the hydraulic dredge and floating pipe discharge for distances from 1 000 to 2 000 ft. I think the pipes used were larger than ordinarily used here, say, about 18 ins. in diameter.

F. S. Washburn, M. Am. Soc. C. E.—The author speaks of 40 to 60 ft. as about the average to which the material was lifted in using ordinary dredges and cars. Of course that is an objectionable feature, but the cost of an extra foot in height in excavating material after it is once gotten under way, is in many instances inappreciable. I have endeavored to estimate, as I have been sitting here, listening to the paper, about what it would cost, under ordinary conditions of dredging, to raise 1 cu. yd. of material that extra 40 to 60 ft., and it is somewhere between $\frac{1}{3}$ and $\frac{2}{3}$ cent a cubic yard. There may be advantages in the ordinary type of dredge, compared with which this extra cost of handling material may be relatively unimportant. In a number of instances I have known devices argued against and thrown aside because the amount of actual mechanical work necessary was greater than in some other method with which it was compared, but which possessed practical advantages entirely outweighing its low efficiency. The point I make is that, while the mechanical work may be much greater, the actual extra cost due to the extra work may be of no consequence, or very small.

R. S. Buck, Assoc. M. Am. Soc. C. E.—Since this discussion has developed the question of the relative efficiency of the different kinds of dredges, I would like some gentleman to give us some information as to the relative efficiency of the dredges as actually lifting the material from place. When I was on work in Vicksburg Harbor under the Mississippi River Commission some time ago that question arose, particularly as to the comparative efficiency of the endless-chain and the clam-shell dredges, and while I had at the time no fair means of getting at the efficiency of the endless-chain dredges, I got pretty well what was the actual cost of raising material from place, and depositing in the river again by means of dumping scows, at an average distance of about a mile and a half. I found the actual cost was about 5 cents per cubic yard. The average depth of water, about 35 ft., I think; the cost was about 5 cents per cubic yard, including towing.

I would like to know, as a matter of curiosity, what the cost of dredging under known conditions with the endless-chain dredger is.

Mr. Crowell.—I happen to know of a case where 10 cents per cubic yard was said to be the price paid to a contractor using the endless-chain dredge. The endless-chain, I think, is more economical than the dipper dredge in working large prisms. There is a point where the dipper dredge or clam-shell dredge begins to lose its economy very rapidly as the depth increases; but in moderate depths, and where the operation is simply to lift the material upon scows, I believe the large-sized dipper dredge will do the work for the least money.

Mr. Gosling.—Is it not more difficult to get an even bottom with the dipper dredge than with the endless-chain dredge?

Mr. Crowell.—Yes, sir.

R. L. Harris, M. Am. Soc. C. E.—In counting that cost of dredging at 5 cents, did you take into account the deterioration of plant, the interest on the cost of outfit, and the item of lost time?

Mr. Buck.—No, sir; I did not count the deterioration; merely the cost of operation.

Mr. Harris.—Neither the deterioration, nor interest, nor loss of time?

Mr. Buck.—That included repairs; it was simply the cost of operation.

Mr. Harris.—I may say that the author speaks of 863 000 cu. yds. and 557 000 cu. yds. having been deposited from the trestles, and then the pumping business was begun. Now, the original idea of one of the

figurers on that work was the pump and the figures he made—14 cents—were almost identically those of the actual cost by pump.

CHARLES B. BRUSH, M. Am. Soc. C. E.—That was before any work was done at all?

Mr. HARRIS.—Yes, sir; at the time of the original bidding for letting.

Mr. BRUSH.—After that, the plan was changed and the trestle was built.

Mr. HARRIS.—There was no limit, as I remember, to what plan could be used. One of the figurers figured entirely on the use of pump.

Mr. BRUSH.—Was the work let out by contract originally?

Mr. HARRIS.—That is my recollection of it.

Mr. BRUSH.—Then the lower bid was thrown out?

Mr. HARRIS.—I do not say anything about bids; I say this was the estimated cost by one of the figurers.

Mr. R. S. BUCK (by letter).—Since the date of the discussion of Col. Hains' paper, I have received the following letter concerning cost of dredging in Vicksburg Harbor.

"UNITED STATES ENGINEER'S OFFICE,
Vicksburg, Miss.,
JANUARY 2D, 1894.

Mr. R. S. BUCK.

Dear Sir,—In answer to your favor of the 27th of December, requesting information in regard to cost of dredging in Vicksburg Harbor, I have the following to state: I find in my report to Captain Rossell of May 2d, 1888, the cost of operating the contractor's dredge *Herndon* (clam-shell bucket), to be for 24 hours:

16 men (laborers), each, $30 per month	$16 00	
2 engineers, each, $60 per month	4 00	
2 captains (diggers), each, $125 per month, ...	8 25	
2 cooks, each $45 per month	3 00	
Total—dredge		$31 25
Tug, 6 men, each $30 per month	$6 00	
" 2 captains, each, $125 per month	8 25	
" 2 cooks, each, $45 per month	3 00	
" 2 engineers, each, $60 per month	4 00	
Total—tug		21 25
Subsisting 36 men (including two U. S. Inspectors), at 50 cents		18 00
Coal for dredge and tug, 6 tons per 24 hours at 40 cents per box		25 00
Incidentals, repairs, etc.		10 00
Total cost		$105 50

An average month's excavation (double turns) was 55 529 cu. yds., (scow measurement), giving a little more than 5½ cents per cubic yard as actual cost of dredging. This does not include interest chargeable to money invested on plant or cost of getting to working locality. An estimate made in 1883, in Captain Marshall's time, gives the same results.

Yours truly,

H. St. L. Coppée."

I would like to add further, that I furnished the data for this estimate, which, on account of the exact rate of wages paid all the men and the exact cost of subsistence not being known, is only an approximation; but the tendency of the figures given is undoubtedly to make the cost of operation appear somewhat larger than was actually the case.

Now, valuing the entire plant at $50 000, which is high, interest at 6% would be $8 22 per day. A considerable portion of depreciation is included in the item in Mr. Coppée's letter, "Incidentals, repairs, etc."; but taking this at 10%, it gives $13 70 per day. These items with the $105 50 accounted for in Mr. Coppée's letter will give a total daily cost of operation of $127 42, or a cost per yard of something less than 7 cents. This does not account for towing to and from place of operation, which item cannot well be estimated in a way to aid comparison.

The digging in Vicksburg Harbor might be called "fairly good." The bucket was rarely heaped, and some very tough material and very large logs were encountered and removed. The *Herndon* was ably handled and worked with great energy.

I think the above will furnish fair means of comparison as far as the clam-shell dredge is concerned, and I believe it would be of general interest if similar data could be had as to the performance of dredges of other types.

P. C. Hains, M. Am. Soc. C. E.*—It is proper to say that the hydraulic dredges used on the reclamation of the Potomac Flats were not designed by me, and I claim no credit for their efficient working and am not responsible for defects. That there were many defects in the machinery is unquestionable. But the particular interest that attaches to this work lies in the fact that it was the first application

* Lt.-Col. Corps of Engrs., U. S. A.

known to me of the hydraulic dredge to a work of magnitude. It was not the first application of such a dredging machine, for Colonel G. H. Mendell, of the Corps of Engineers, had one in operation at or near Oakland, Cal., before the work on the Flats was begun.

The question of reducing the number of angles or bends in the pipes, as well as the necessity of devising some better means of connecting them, was a frequent topic of discussion between me and the contractor. I do not regard the hydraulic machine used by us on the Potomac as the perfection of hydraulic dredge. I think there are many things in connection with it that are susceptible of improvement, and as the machine comes more generally into use, improvement will follow.

The velocity of the water flowing through the pipes was never determined with any degree of accuracy. It was rather roughly estimated from observation and the known capacity of the pump working under certain known conditions.

In regard to the prices of the material handled twice by the dredges and once by the cars, and leveled by the water-jet, it should have been stated that 21.2 cents per cubic yard was the price paid by the Government for the dredging and depositing of the material in place at the required grade. It included the cost of the trestle, cars, locomotives and everything, including the profits to the contractor, if there were any. What I meant to call attention to was, that the trestle alone cost the contractor, not the Government, about two cents for every cubic yard of material he handled, the amount of material handled being nearly 1 500 000 cu. yds., and the cost of the trestle about 3 000 000 cents ($30 000). With this explanation the apparent error referred to by Mr. Crowell disappears.

As to the comparative cost of dredging by the hydraulic machine or by the ordinary dredging machine in common use, such as the clamshell and dipper dredge, it is impossible to say what that will be until a full understanding of the conditions is assured. The ordinary clamshell or dipper dredge is adapted to many varying conditions; the hydraulic machine is not. On the Potomac Flats the material had to be taken out of the bed of the river, transported from 1 000 to 5 000 ft., and then deposited in places where it would be useful as earth filling. It could not be carried in scows to the places of deposit, for there was not water enough to float them, and most of the material had to be de-

posited above the level of the water in the river. Some means of transporting the material had to be improvised, and the use of water flowing through pipes at a high velocity was the most economical.

To make a fair comparison between the cost of the two kinds of dredging one must know exactly the character of the material, the depth of cutting, the exposure of the locality to storms and waves, the distance to which the material, after being dredged, has to be taken, and the relative height of the place of deposit. If it be a simple case of dredging and depositing the spoils in a scow, the ordinary clam-shell or dipper dredge would give the most economical results as a rule, but not always. Much will depend on circumstances, such as the exposure of the locality and the character of the material. The sandy bars at the mouths of rivers exposed to the open sea are generally so rough that a hydraulic machine provided with a flexible pipe reaching to the bottom gives far the best results. At the entrance to New York Harbor, the hydraulic machine was eminently successful, and the ordinary clam-shell or dipper dredge a failure; so, also, at the mouth of the St. Johns and Cape Fear Rivers, and other places. To institute a fair comparison between the two methods, the conditions must be taken into consideration. In the case of the Potomac Flats the conditions were peculiarly favorable to the hydraulic machine.

The point made by Mr. Washburn is well taken, that, while the mechanical work may be much greater, the actual cost due to the extra work may be of no consequence, or very small. That is just what operates in favor of the hydraulic dredge. The extra mechanical work of lifting and forcing an immense quantity of water through pipes is great, but the extra cost is comparatively small.

I may also add, that in offering the work to contractors, the method of doing it was always left to be stated by the contractor himself, it merely being required that the method should be practicable and adapted to its accomplishment. The first contract was awarded at 21.2 cents per cubic yard, scow measurement, that being the price given by the lowest responsible bidder. There was one other, and only one other, bid made at the time that gave a lower price, and that was 19¼ cents per cubic yard. The bid, however, was informal and defective, the bond worthless, the bidder without the means of doing the work in any manner whatever, and not responsible. That bid was accordingly rejected.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

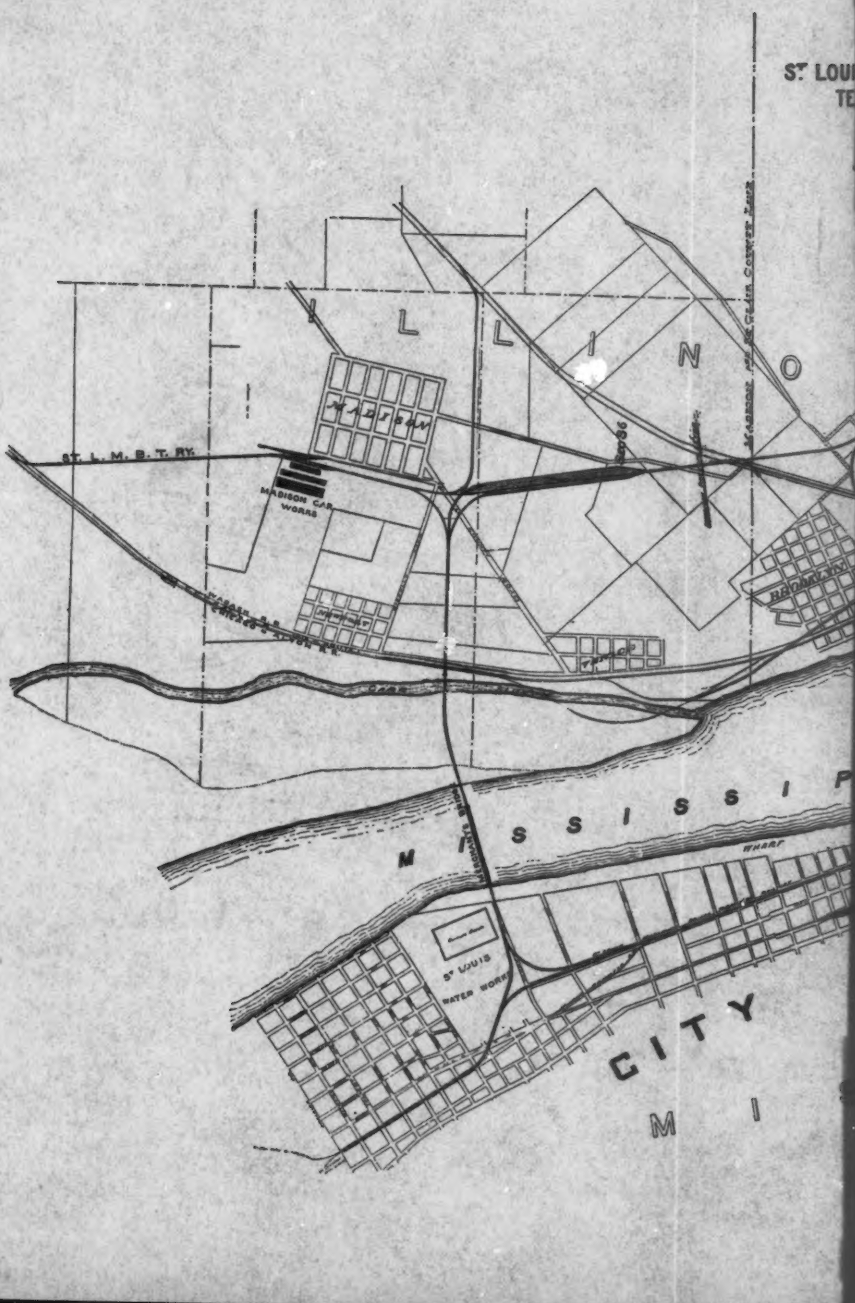
705.
(Vol. XXXI.—May, 1894.)

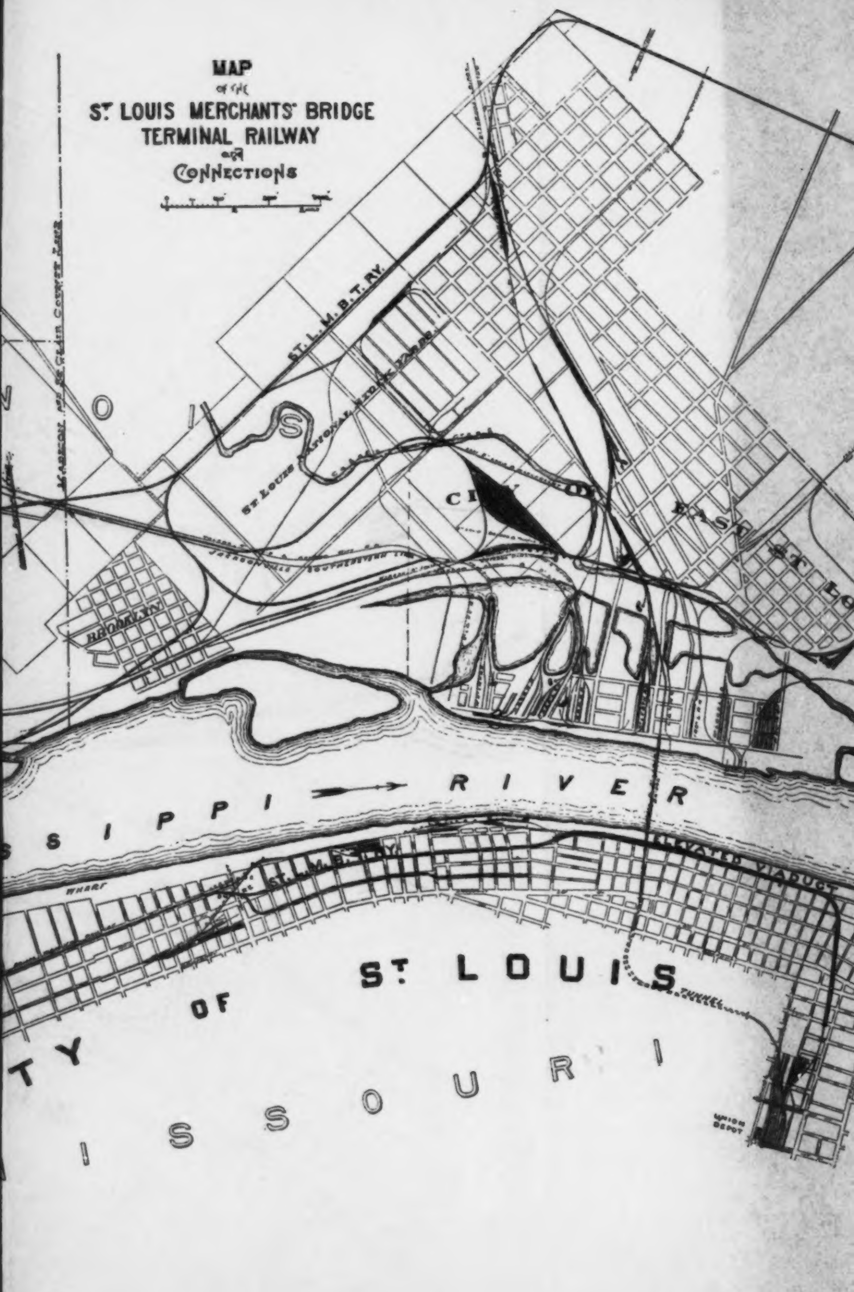
THE MERCHANTS' BRIDGE TERMINAL RAILWAY VIADUCT AT ST. LOUIS, MO.

By ROBERT MOORE, M. Am. Soc. C. E.

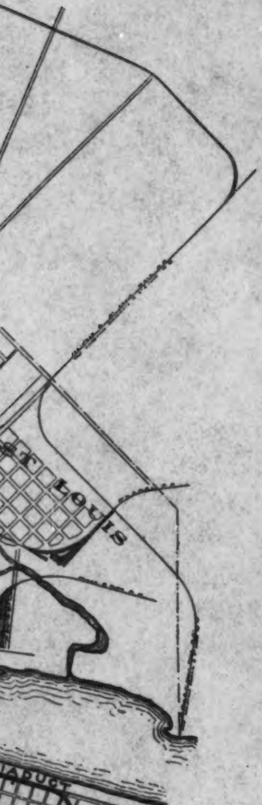
In developing a new system of terminal tracks designed to unite the various railway systems of St. Louis with the Merchants' Bridge over the Mississippi River it became the duty of the writer as Chief Engineer to build a section of elevated railway which it is the purpose of this paper to describe.

This structure was authorized by Ordinance 14 078, approved July 9th, 1887, and had for its special object to reach the south end of the city and the railways in the Mill Creek Valley. As will be seen by the accompanying map (Plate LXIII), it extends from the north side of Carr Street, over which it passes, through city block 18 to the river front, along which it runs southwardly for more than a mile to Poplar Street. Here it curves sharply to the west and passes through the blocks to the west side of Seventh Street about 100 ft. north of Gratiot Street, where the permanent iron structure ends. Beyond this, 150 ft. of temporary wooden trestle brings the track to the surface of the ground, and in a few hundred feet more it is connected with the tracks of the St. Louis and San Francisco and Missouri Pacific railways. The total length of iron structure is 8 176$\frac{7}{10}$ ft., or about 1$\frac{55}{100}$ miles.


ST. LOU
TE
ILLINO
MADISON
ST. L. M. B. T. RY.
MADISON CAR WORKS
BROOKLYN
MISSISSIP
WHARF
ST LOUIS WATER WORKS
CITY
MI


MAP
of the
ST. LOUIS MERCHANTS' BRIDGE
TERMINAL RAILWAY
and
CONNECTIONS
ST. L. M. B. T. RY.
ST. LOUIS NATIONAL STOCK YARDS
BROOKLYN
EAST ST. LO
R I V E R
S I P P I
ELEVATED VIADUCT
ST. L. M. B. T. RY.
WHARF
ST. LOUIS
OF
TY
TUNNEL
UNION DEPOT
S O U R I

LATE LXIII.
. SOC CIV. ENGRS.
XXXI, No. 705.
S' BRIDGE TERMINAL RAILWAY.



The preliminary plans were made during the summer of 1889, and in November of the same year a contract for the construction of both foundations and superstructure was made with the Phœnix Bridge Company. Work was begun in December, 1889, and the whole structure completed, excepting the painting, in March, 1891, about fifteen months from the time of beginning. This was longer than would have been necessary but for delays in securing the right of way, such as appear to be almost unavoidable in works of this kind—delays which were a source of embarrassment to the engineer as well as to the contractor. To avoid enhancing the price of real estate needed by the railway company, which would have resulted from the appearance on the ground of an engineering party, no such party was put into the field until all property that could be bought at private sale had been secured. The preliminary plans had, therefore, to be made from such imperfect data as could be obtained from city maps without a survey. In some cases exact lengths of girders could not be safely given until buildings which stood in the way had been torn down so as to allow of actual measurements on the ground. Even where there was no lack of exact knowledge, the fact that the contracts for right of way nearly always contained some stipulation as to the location of the columns rendered it impossible to make a final plan until all such negotiations were definitely closed.

The details of grades and curvature are shown on the accompanying profile (Plate LXIV). The sharpest curve is one of 14° (410 ft. radius) for about 260 ft. There is also one curve of 12° (478 ft. radius) for 160 ft., and one of 10° (574 ft. radius) for 60 ft. The remainder range from 2° 45′ down to 1°. All the longer curves are connected to the tangents at each end by spiral transition curves. The maximum rate of grade is $\frac{8}{10}$ of 1%, or $42\frac{24}{100}$ ft. per mile. On curves the grade is reduced half a tenth per 100 ft. per degree of curvature, and all changes of grade are made by vertical curves. Going southwardly the length of descending grades amounts to $28\frac{4}{10}$%, the level grades to $28\frac{6}{10}$%, and the ascending grades to 43%, of the whole line.

The structure itself consists of four lines of plate girders carrying two lines of railway track. With the exception of twelve masonry piers all the supports are steel columns built of Z bars or of plates and angles. The two rows of columns are directly in line with the two outer girders. This brings them $18\frac{1}{2}$ ft. apart between centers and allows just space

enough for two lines of teams, thus causing the least possible interference with the traffic on the street. The height and section of the column is made such as to allow both the longitudinal and the cross girders to be riveted to them directly, which is done everywhere except at expansion joints and in a few cases where special conditions made it possible. To further stiffen the structure alternate bents are braced, where it is practicable, as in passing through the blocks, by struts near the ground and diagonal rods. When this method would interfere with the street traffic a partial bracing is substituted, consisting of latticed struts brought down to a line 14 ft. above the pavement.

The column foundations are all formed of Portland cement concrete surmounted with granite cap stones. Imbedded in the concrete and extending through the cap stones are anchor rods to secure the columns to the foundations. In those parts of the structure where the bending stresses are taken up by bracing brought down to the ground, the only duty of the anchor rods is to prevent displacement at the foot of the column, for which light rods only are needed. Those used are $1\frac{1}{4}$ ins. in diameter, 3 ft. long, two to each column. When the bracing is not brought down, the bending stresses are transmitted to the anchor rods, and they were therefore proportioned with a view to holding the column at the bottom rigidly "fixed ended." For this purpose four rods $1\frac{5}{8}$ ins. in diameter, with upset ends, were used for each column. Their lengths was usually $7\frac{1}{2}$ ft., though in special cases they were made longer. In deference to the common fear of the injurious effect of cutting screw threads in steel, the anchor rods were made throughout of iron. The result was not satisfactory. Upon testing full-sized rods, all of them broke in the screw thread, showing, apparently, a weakening of the metal in the process of upsetting. After annealing, however, their strength was so far restored that, upon further tests, they broke in the body of the bar. They were, therefore, accepted and placed in the structure. Two of them, however, were broken while in position during construction, and were the only failures which at any time occurred. Had they been of steel, this fact would have been accepted as a sufficient explanation of the failure.

In placing the anchor rods use was at first made of wooden measuring rods furnished by the Phœnix Bridge Company, and certified by them to be in exact agreement with their shop standards. It was soon found, however, that the practical difficulty of making and main-

PROF

OF TH

MERCHANTS BRIDGE TERMI

AT

ST. LOU

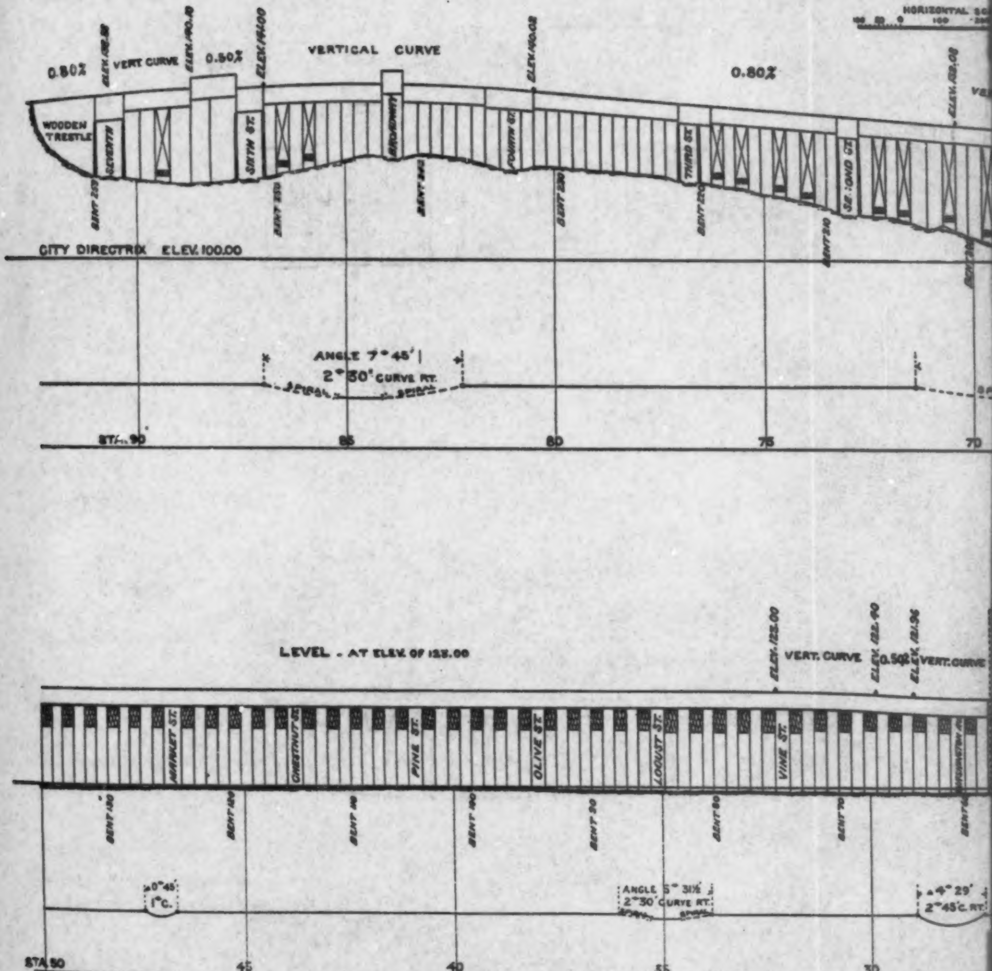


PLATE LXIV
TRANS. AM. SOC. CI
VOL. XXXI, No.
MOORE ON MERCHANTS' BRIDGE

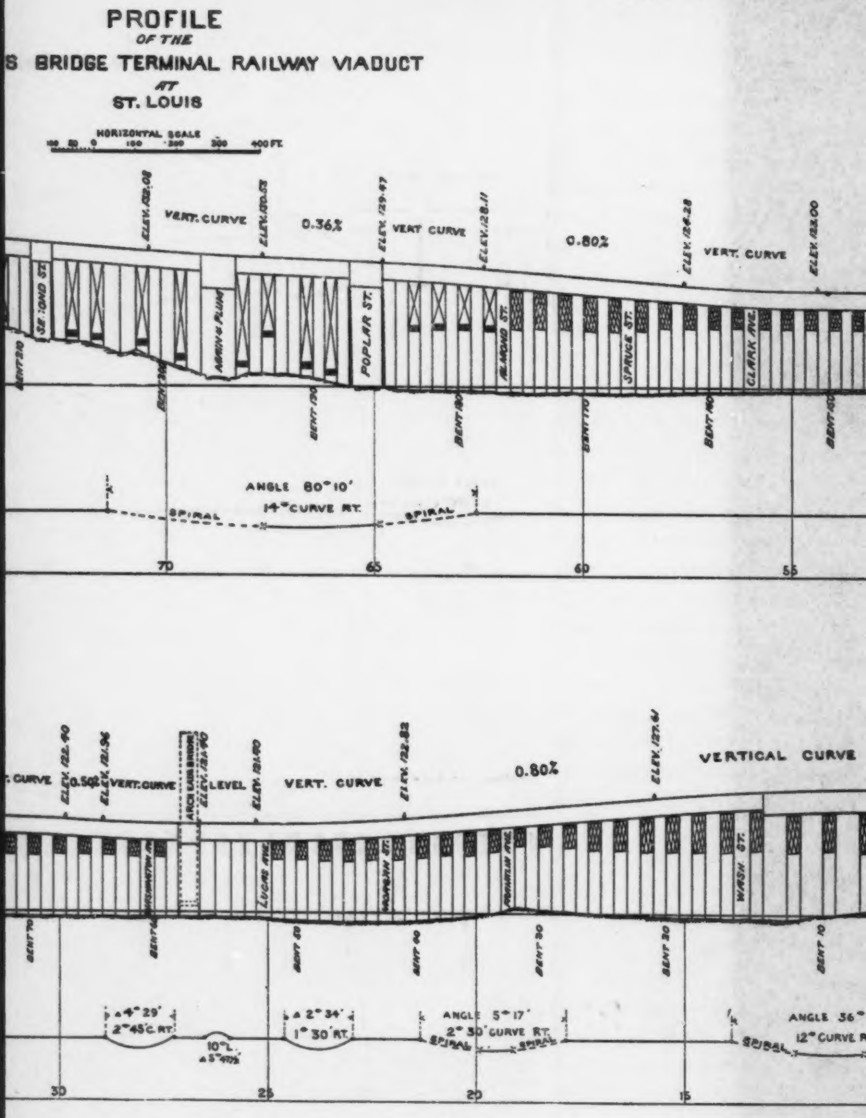
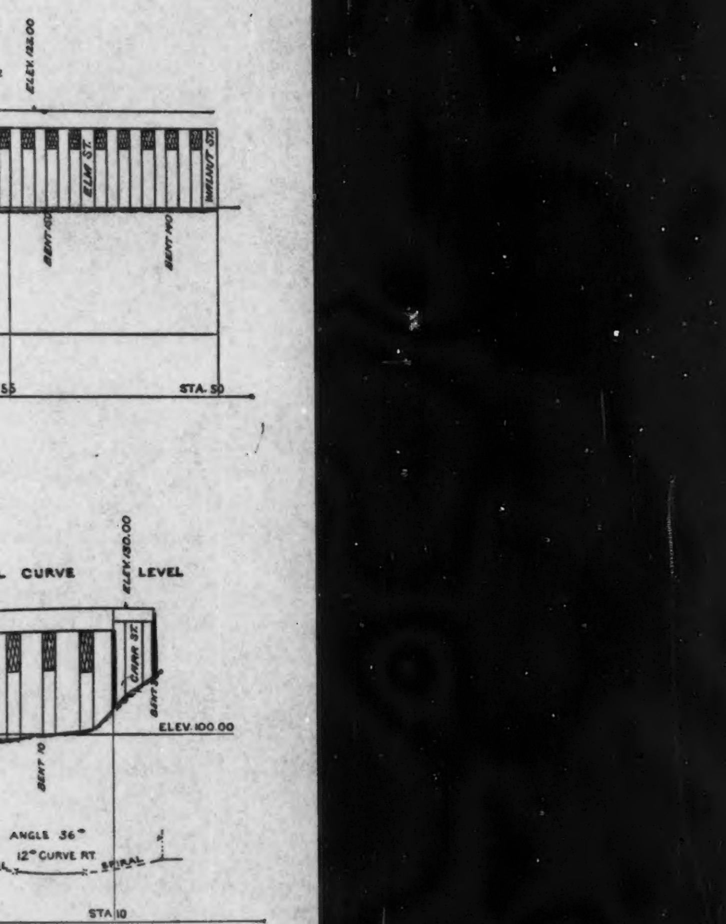


PLATE LXIV.
AM. SOC. CIV. ENGRS.
. XXXI, No. 705.
ITS' BRIDGE TERMINAL RAILWAY.



taining perfect contact between the rods and of keeping them at all times level was so great as to make the results less trustworthy than those obtained with a steel tape, which was thereafter used to the exclusion of everything else. The results thus obtained checked so well with the shop measurement that in setting nearly 2 000 anchor bolts for more than 1 200 longitudinal and cross girders no misfits of any consequence occurred.

The material encountered in the foundations was mainly a firm clay upon which a maximum load of 3 000 lbs. per square foot including the live load, was placed. Ordinarily a much greater load no doubt could have been safely borne, but the fact that in times of freshet all the foundations on the river front are under water and liable to saturation made it desirable to keep the loading as low as reasonably practicable. In some cases where quicksand or soft mud was encountered piles were used, over which the concrete was laid without capping, as shown in the plans.

In eighty-three cases the foundations were placed directly upon the rock. In a few of these the rock was found so near the surface that holes had to be drilled to receive the anchor bolts and the question arose as to how to fasten them so as to develop the strength of the bolts and obtain an anchorage equivalent to that of the block of concrete used in other cases. To solve this question two sets of experiments were made. In the first, a number of holes were drilled in a solid ledge near the line of the work, in which holes were inserted rods fastened by means of wedges, iron cement or "rust joint," sulphur, lead and Portland cement used neat and also mixed with sand. By means of an I beam arranged as a lever and weights of known value, a pull was then applied to each bolt until it was either pulled out or broken off. As the result the bolts fastened with wedges and with iron cement were all easily pulled out. Of those fastened with lead and sulphur, each melted and poured in around the bolt, two out of three were broken and one pulled out. Of those fastened with neat cement five out of six were broken and but one pulled out. The two bolts fastened with a mortar of cement and sand in equal parts were both broken.

These results showed so well for the cement anchorage that another series of tests with neat cement alone was made at the Washington University testing laboratory. In this series holes were drilled in limestone blocks 10 x 10 ins. square and 12 ins. deep, this being as

large blocks as could be got into the testing machine, and in these holes, rods of 1 and 2 ins. diameter were fastened with a mortar of neat Portland cement. Half of the rods were plain, the other half had a screw thread running the full depth of the hole. Ten days were allowed for the cement to set, and the rods were then pulled until loosened in the holes or until the stones were broken. The plain rods developed before loosening a resistance of about 500 lbs. per square inch of imbedded surface. The threaded rods showed a still higher resistance, in fact the stones were not strong enough to develop the full strength of the anchorage. To bring out all the facts a further series of experiments with larger stones would be required, but these results were so satisfactory that wherever anchorage in rock was required, threaded rods fastened with neat cement were used.

The concrete used in all foundations was made of Portland cement, river sand and broken limestone in the proportions of 1 part of cement to 3 of sand and 6 of stone. With these proportions, it was found that one barrel of cement made 0.96 yds. of concrete. To ascertain the strength of this concrete, a number of beams 30 ins. long and 9 ins. square were made from batches mixed for actual use in the work. These were then buried in the ground for periods of 8, 9, 10 and 11 months. At the end of this time they were broken transversely, by means of a lever apparatus so arranged as to give a gradually increasing, but definitely known, load. The results are shown in the table on page 505.

The tensile strength computed from these experiments showed an unexpected uniformity, the average being 354 lbs. per square inch, with a minimum of 307 and a maximum of 412 lbs. The strength of the neat cement from which the concrete was made, as shown by a seven-day test, ranged from 307 to 567 lbs., the average being 432 lbs. per square inch. That is to say, that while the variation in the strength of the neat cement was 85%, the variation in the concrete was only 34 per cent.

With the exception of the anchor bolts and a few minor members, the material used is mild steel with an ultimate strength of not less than 60 000 nor more than 68 000 lbs. per square inch, an elastic limit of not less than 37 500 lbs., and an elongation in 8 ins. of not less than 22 per cent. All rivets were made of soft steel having an ultimate strength of not less than 54 000 nor more than 62 000 lbs. per square

inch, an elastic limit of 33 750 lbs., and an elongation in 8 ins. of not less than 25 per cent. For both grades of steel the usual quenching and bending tests were also specified, but no chemical tests of any kind were required. All sheared plates were required to be planed and all rivet holes reamed.

Cement Tests.				Concrete Tests. Proportions, 1, 3 and 6.							
						Dimensions in inches.					
Brand of Cement.	Time, in days.	Mixed Neat, pounds.	Mixed 1 to 1, pounds.	Number of beam.	Age, in months.	Depth.	Width.	Length.	Breaking load.	Computed tensile strength.	
				1	8	9¼	9¼	27	6 610	338	Pounds per square inch.
A	7	307	195	2	9	9	9¼	27	7 290	394	" " "
A	28	502	440	3	9	9½	9	27	6 520	325	" " "
							Mean value......			352	
				1	9	9¾	9	27	6 490	307	Pounds per square inch.
B	7	503	299	2	8	10	9½	27	8 030	342	" " "
B	28	591	355	3	9	9	9¼	27	7 160	387	" " "
							Mean value......			345	
				1	9	9	9½	27	6 500	342	Pounds per square inch.
C	7	567	320	2	9	9¼	9½	27	7 470	372	" " "
C	28	703	482	3	9	10	9¼	27	7 980	349	" " "
							Mean value......			354	
				1	10½	9¾	9	27	7 610	360	Pounds per square inch.
D	7	350	197	2	10½	9	9	27	6 030	335	" " "
D	28	545	338	3	11½	9	9⅛	27	6 780	357	" " "
				4	11½	9	9½	27	7 820	412	" " "
							Mean value......			366	

The superstructure was proportioned for a dead load consisting of the weight of the iron and steel in the structure itself, plus the weight of the track, assumed as 450 lbs. per lineal foot, and a live load for each track, assumed to be moving in either direction and consisting of two engines coupled, each with its tender weighing 208 000 lbs., being Mr. Cooper's "Extra Heavy A." The longitudinal bracing was designed to resist the force developed by suddenly stopping a train of 3 000 lbs. per lineal foot, headed by two of the engines just mentioned, the coefficient of friction of wheels upon rails being assumed at 20% of the load on the wheel. The wind pressure for computing the lateral stresses was taken at 750 lbs. per lineal foot.

In designing girders, a shearing stress was allowed of 7 000 lbs. and a flange stress of 10 000 lbs. per square inch of net section. Wind bracing and anchor rods were proportioned at 15 000 lbs. per square inch, the stress on anchor rods being increased by 10 000 lbs. for initial adjustment.

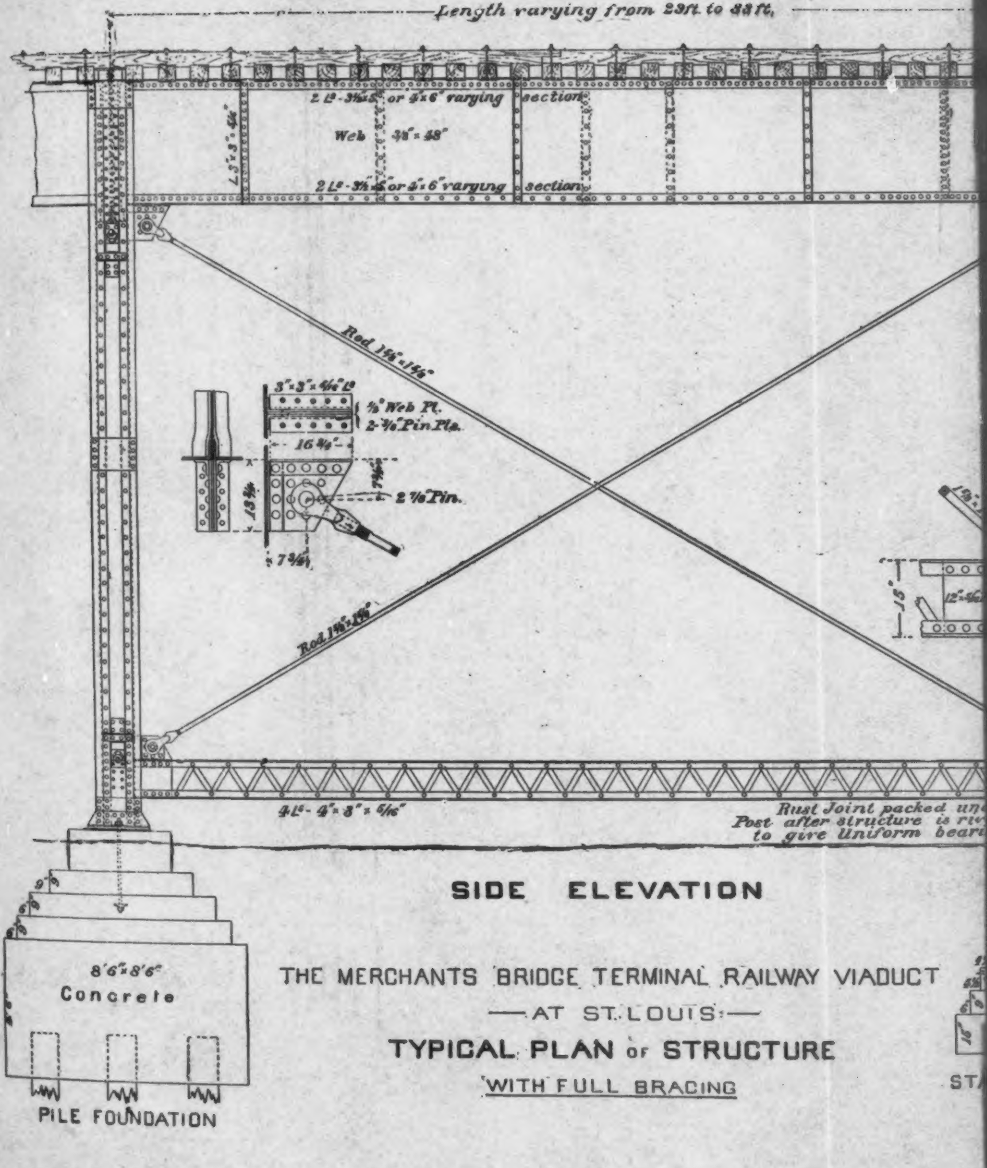
In designing the posts of the partly braced structure, the unit stresses were varied for the three different cases as follows:

First.—For the direct load and bending due to the tractive force of the engine, this being a working condition, a unit stress of 10 000 lbs. per square inch was allowed.

Second.—For the direct load and bending due to wind pressure or a sliding train, an exceptional condition, a unit stress of 14 000 lbs.

Third.—For the direct load combined with bending due to wind and tractive force or a sliding train, also an exceptional condition, 20 000 lbs. In each of these three cases it was found that the amount of metal needed to resist bending was much greater than that required for the direct load. The total weight of iron and steel in the structure is 9 791 000 lbs., or 1 197 lbs. per lineal foot.

With the exception of five or six spans, the whole structure was erected by means of a traveler moving on a temporary track laid on the outer girders. The material was brought to the site of the work by teams and lifted to final position by the traveler. When not delayed for material, six spans were easily erected in one day of 10 hours. Before the work of erection had proceeded far it was found that the bearing of the columns on the granite cap stones of the foundation was not as true and uniform as was desired. This was due sometimes to the fact that the cap stones were not a perfectly level plane, and at other times to the fact that the base plate of the column was not a true plane, or not exactly perpendicular to the axis of the column. As this want of uniform bearing would develop unequal reactions at the base and eccentric stresses in the column itself, it was necessary to correct it. After unsuccessful experiments with Portland cement and with lead, it was found that by far the best method was to lift the column slightly by wedges, say ¼ in., and then fill in the space thus left between the cap stone and the base of the column with iron cement or "rust joint." By using long blades of thin steel and borings of cast iron, this was easily done, and when well done gave an entirely satisfactory result. Upon taking down a post, referred to



SIDE ELEVATION

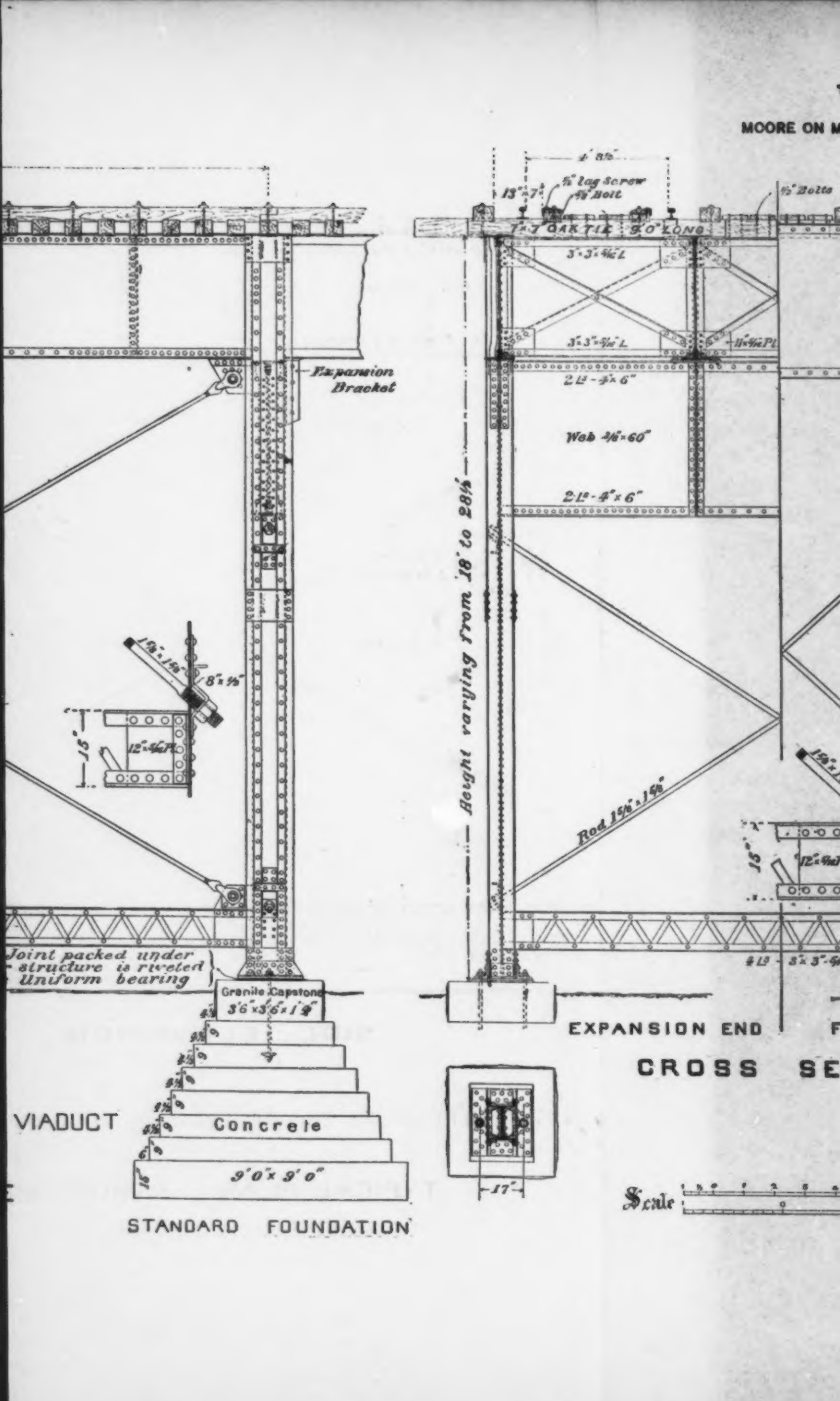
THE MERCHANTS BRIDGE TERMINAL RAILWAY VIADUCT

—AT ST. LOUIS—

TYPICAL PLAN OF STRUCTURE

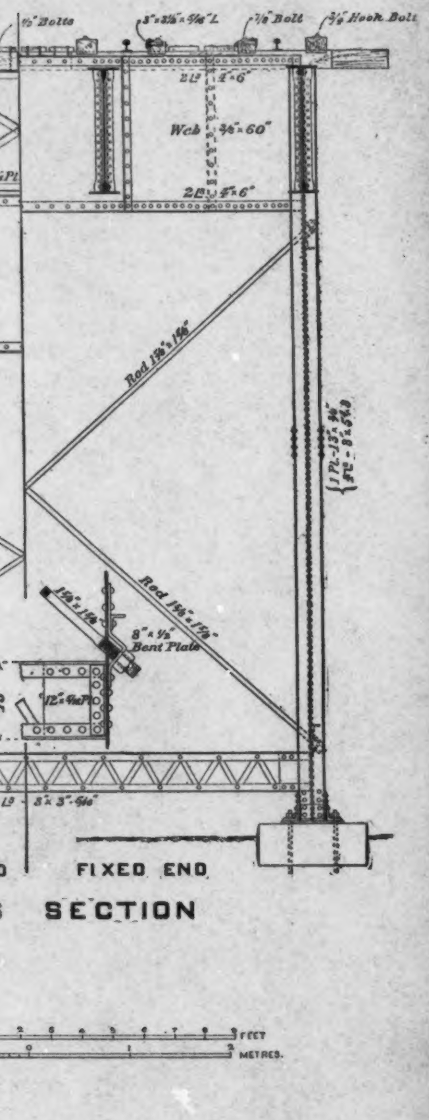
WITH FULL BRACING

ST



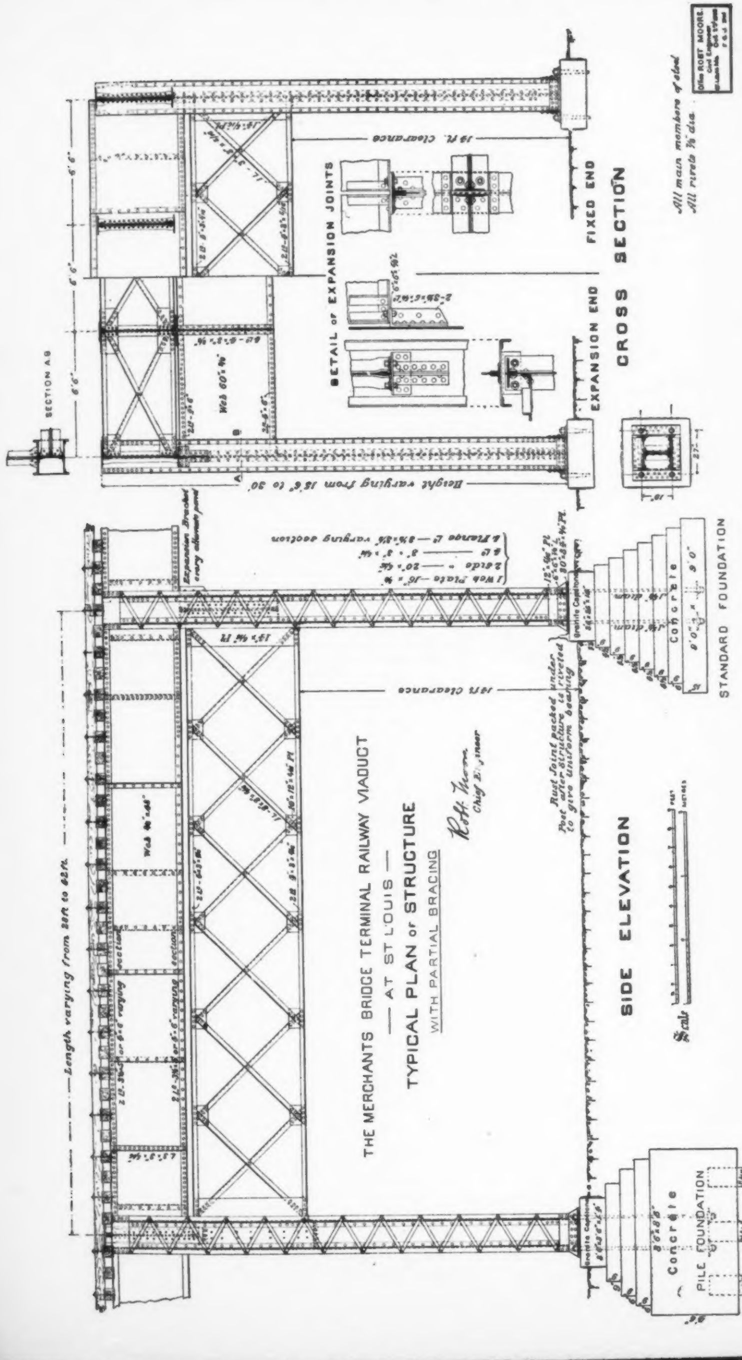
MOORE ON M
7×7 OAK TIE 9'0" LONG
Expansion Bracket
Web 5/16×60"
2 Ls-4"×6"
Height varying from 18' to 28½'
Rod 1⅝"×1⅝"
Joint packed under structure is riveted Uniform bearing
Granite Capstone
3'6"×3'6"×1'4"
Concrete
9'0"×9'0"
VIADUCT
STANDARD FOUNDATION
EXPANSION END
CROSS SE
17"
Scale

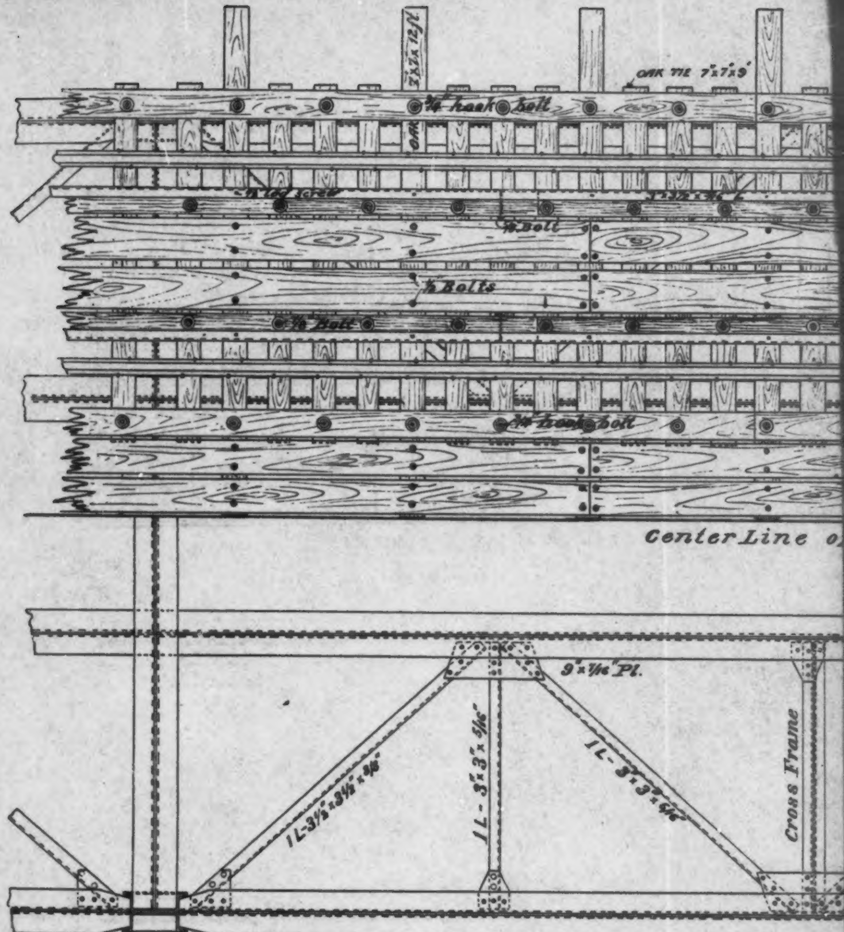
PLATE LXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 705.
OORE ON MERCHANTS' BRIDGE TERMINAL RAILWAY.



SECTION

PLATE LXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 705.
MOORE ON MERCHANTS' BRIDGE TERMINAL RAILWAY.



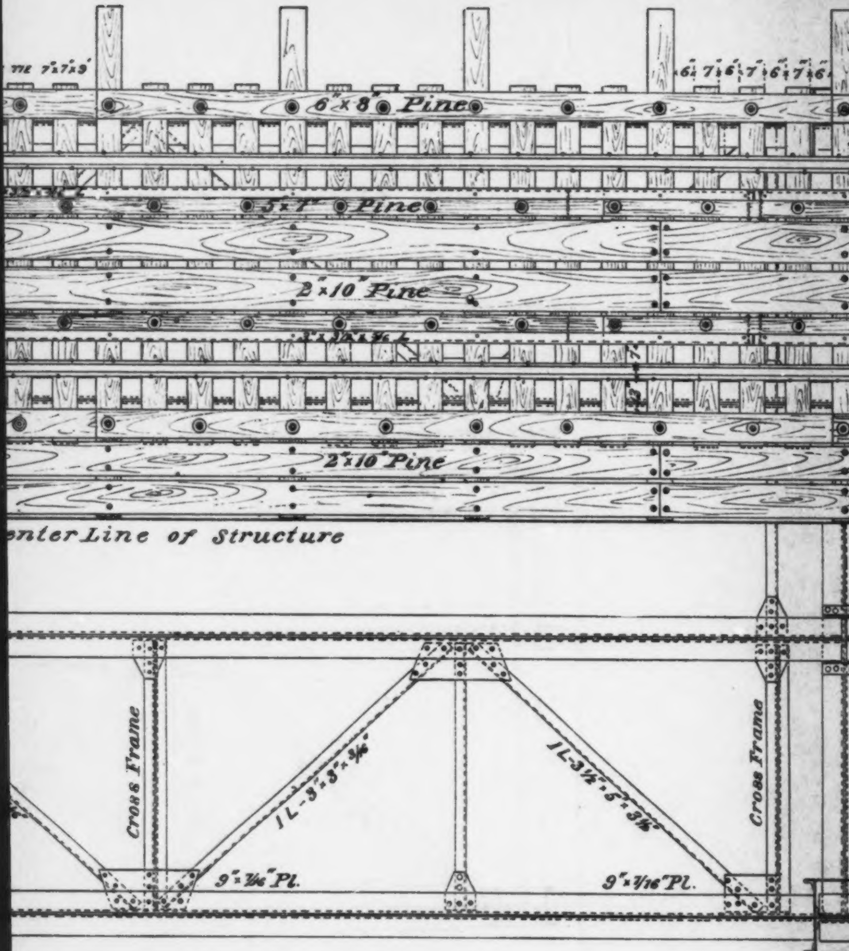


THE MERCHANTS BRIDGE TERMI

— AT ST. LO

FLOOR PLAN AND LAT

SCALE IN FE



DGE TERMINAL RAILWAY VIADUCT

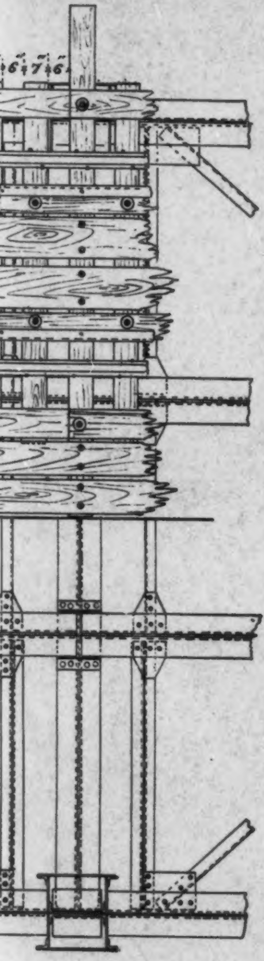
AT ST. LOUIS —

AN AND LATERAL BRACING

SCALE IN FEET

Robt. Moore.
Chief Engineer

PLATE LXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 705.
ERCHANTS' BRIDGE TERMINAL RAILWAY.



...e.
...gineer

Office ROBT. MOORE.
Civil Engineer.
St. Louis Mo. Oct. 2nd 1893
F. G. J. Del.

hereafter, in July, 1893, after three years' service, the cement joint was found to be well distributed and very hard. The result was, in fact, better than would have been attained by efforts to secure perfect workmanship in cutting and setting the stones, or absolute accuracy in the construction of the columns. As often occurs in practice, it was better to provide an adjustment than to waste time in striving for an impossible accuracy in original construction. In doing the work over again, the writer would make this adjustment a part of the original plan.

In making the iron cement it was found important to use only a minimum quantity of sal ammoniac, and to avoid the use of sulphur altogether, thus confirming the experience already gained on the New York elevated lines.* The formula may be stated thus:

Dissolve 1 oz. of sal ammoniac in 1 gall. of water. Take perfectly clean borings of cast iron and mix with a minimum amount of the above solution. Use as soon as it begins to warm up.

The floor is made of 7 x 7-in. white oak ties, spaced 13 ins. between centers and notched over the girders, to which they are further secured by hook bolts passing through the outer guard rails. The inner guard rails are placed 7 ins. clear from the guard side of the rail, or far enough to allow a derailed wheel to drop on the ties. Both inner and outer guard rails are of white pine, but the inner are shod with 3 x $3\frac{1}{2}$-in. angle iron, to protect them from abrasion in case of derailment. One tie in every four was made 3 ft. longer than the intervening ties in order to form a support for a plank footwalk outside of each track and also between them. Only the inner footwalk was actually laid at the outset, but the outer walks with a hand-rail have since been found a matter of necessity for the protection of the men and the convenient operation of the road. This conforms to experience elsewhere, and, were the writer to do it over again, he would make both the inner and outer footwalks with a permanent hand-railing a part of the original construction.

On curves the outer rail was elevated by means of longitudinal wooden strips laid on top of the outer girder in a channel formed by pieces of angle iron riveted to the top flange. This allowed the use of ties of uniform thickness notched in the usual manner, and simplified materially the work of placing them. The rails on curves,

---

*See Trans. Am. Soc. C. E., Vol. X, p. 124.

in addition to being spiked, were further secured to the ties by Bush interlocking bolts, using three or four pairs to each rail. The results were very good, but the writer would now add a tie plate and would probably use screw spikes of the type in common use in Europe, as being cheaper than the interlocking bolts, and perhaps equally effective, and would discard the ordinary spikes altogether as unfitted for heavy service. In fact, their complete abandonment for any service can only be a matter of time.

Under the head of painting, the specifications required first a coating at the shop with linseed oil, and, second, a painting after erection, with two coats of paint of a quality and color to be approved by the engineer. The paint used was an iron-ore paint brought in the form of paste to the work and then mixed for use with linseed oil. Several ready mixed paints were found upon examination to contain so much benzine that all were rejected. One paint of this class, which was strongly urged by the manufacturers as "an exceptionally good paint for iron work," was found to contain no linseed oil at all. As a basis for future comparison, part of the structure was painted with red lead, and in a few years definite results of permanent value will be obtained.

Since its completion, the structure has borne the test of actual use very satisfactorily. It is very stiff, and, with a single exception, has so far required no alteration or repairs of any kind. The exception referred to was the removal of a column damaged by a collision with a Missouri Pacific freight train August 18th, 1892. This train, composed of loaded box cars, was backed northwardly on a surface track and was derailed by a misplaced switch about 100 ft. south of Post 95A. Though struck with a direct blow sufficient to turn the cars over at right angles to the structure and bring them to rest, the post was not disabled. The angle and side plate which received the blow were sheared off for several inches near the bottom, and the other side plate and angle considerably bent. But the foot of the post was not moved, and the post itself was continued in service for 11 months, or until July, 1893, when it was replaced. Though a somewhat costly experiment to the company owning the cars, to the company owning the structure and to the engineer it gave a proof of the toughness of the metal and the firmness of the anchorage that was very satisfactory.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

706.

(Vol. XXXI.—May, 1894.)

THE RELATION OF WHEELS TO FROG POINTS AND TO GUARD RAILS.

By ARCHIBALD A. SCHENCK, M. Am. Soc. C. E.

READ APRIL 18TH, 1894.

WITH DISCUSSION.

The short paper presented herewith is a graphical examination, by means of full-size drawings, of the relation of wheels to frog points and to guard rails. It is chiefly the report made by the writer, as Chief Assistant Engineer of the New York Central and Hudson River Railroad Company, in pursuance of instructions from the Chief Engineer. The rail shown in the drawings is the standard 80-lb. rail of that company. The Chief Engineer, Colonel Walter Katté, suggested the presentation of the paper to the Society, as likely to have interest for its members. It will perhaps call attention more urgently to the desirability of joint action on the part of the Society with other associations to secure proper relations between wheel gauges and track superstructure. It is not possible in a general meeting, and with limited time, to present the subject in an exhaustive manner, and this paper is intended merely as a suggestion for action by the Society, only a sufficient number of facts being presented to exhibit to the Society the desirability of some action.

Two sets of maximum and minimum wheel gauges (back to back of wheels) have in past years been adopted by the Master Car-Builders' Association (see Fig. 1). The first set, in 1885 (see Figs. 2 to 5, Plate LXVIII), gave for *A B*, Fig. 1, 4 ft. 5½ ins. as the maximum, and 4 ft. 5¼ ins. as the minimum, allowed. In 1887 a larger maximum, 4 ft. 5¾ ins., and a smaller minimum, 4 ft. 5 ins., were adopted, and have since been maintained, giving the large allowed variation of ¾ in. This action appears to have been taken without concurrent action by any other large body of officials or engineers in charge of track. It leaves many thousand cars in this country running over frogs and track unsuited to this later or 1887 set of wheel gauges.

Some articles have appeared from time to time, designed to examine this subject entirely or chiefly by means of computations. In this paper computations have been laid aside as much as possible, and an attempt has been made to show, by full-size drawings, as stated, just what the state of affairs is, and how much interference takes place in various cases. There are so many uncertainties as to what the exact dimension are in any case, and

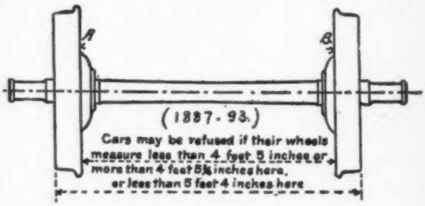


Fig. 1.

the possible starting points for computations are so numerous that the number of computations and of figures soon becomes bewildering, and the mind is led away from definite conclusions. The track gauge point as taken on the wheel is found to lie on a curved surface, without any distinctive feature to identify it. The curves in the flange and tread of the wheel and in the top of the rail head prevent the ascertaining by calculation of any definite point of contact from which to calculate further. The wear of wheel and rail add to the uncertainties of computations. The only method even approximately clear of great possibilities of confusion is to place a full-size drawing of the wheel tread on a full-size drawing of the rail, and ascertain results by following as closely as possible what occurs in actual operating.

Most of the papers that have been written on this subject have

Fig. 5.

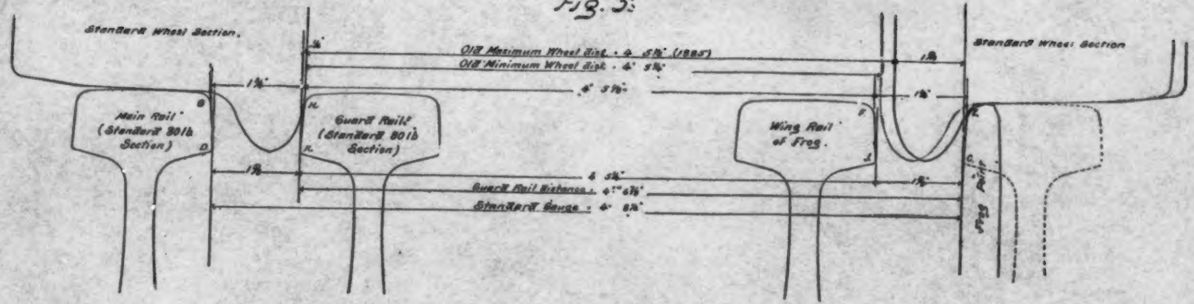


Fig. 4.

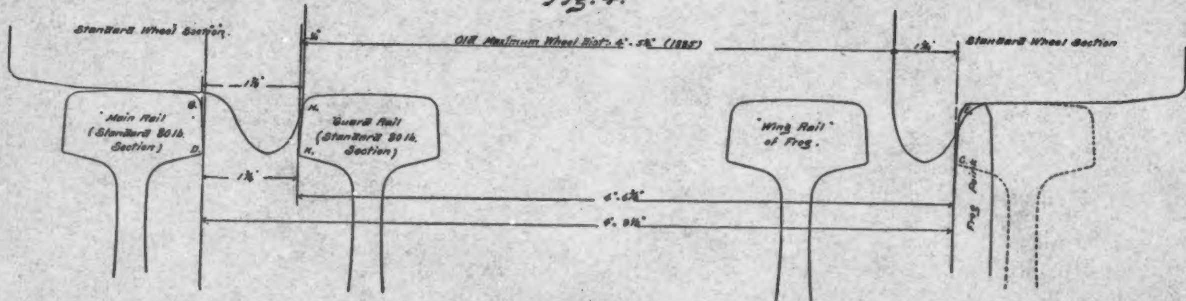
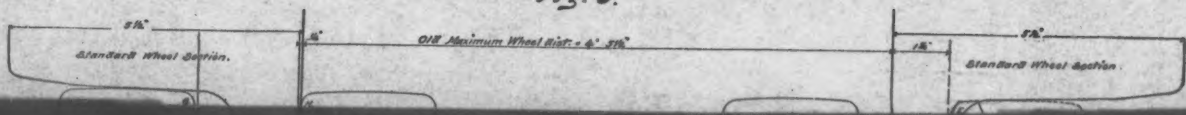


Fig. 3.



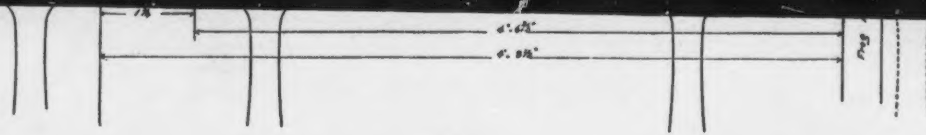


Fig. 3.

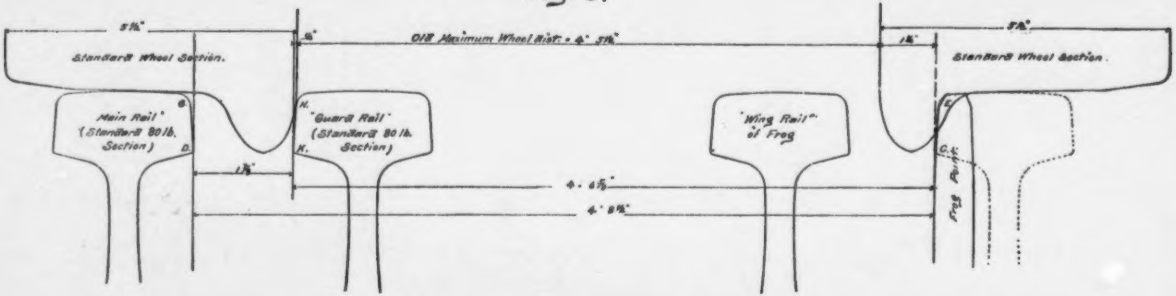
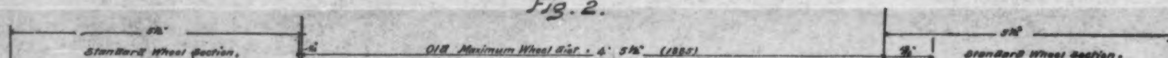
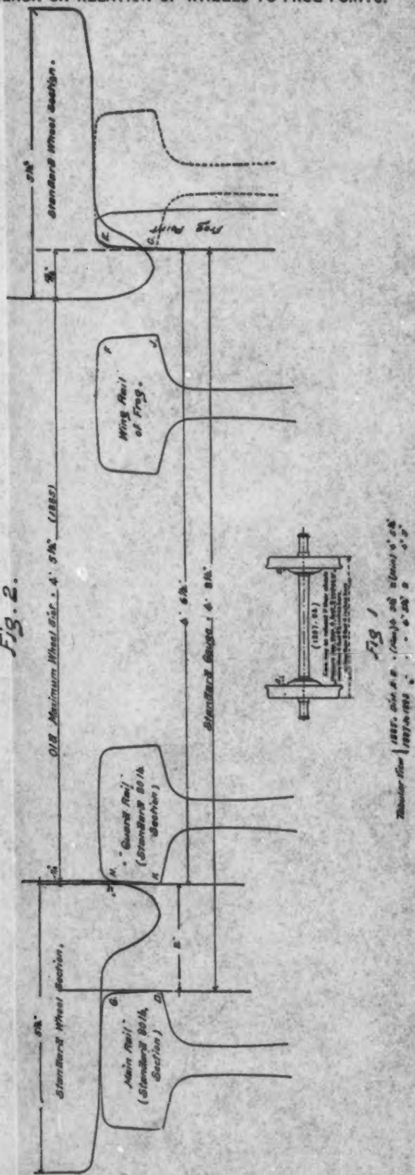


Fig. 2.



SCHENCK C

PLATE LXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.



aimed to determine a flange thickness, and, by adding twice this thickness to the wheel gauge, to compare the resulting figures with the track gauge. In practice, there is no such definite flange thickness. There is an infinite number of flange thicknesses, because the flange is curved. To speak of "the" flange thickness is as indefinite as to speak of "the" chord of a curve, there being an infinite number. With a sloping rail head, a thickness of flange cannot be taken at the height where the track is gauged (the bottom of rail head in most cases), as the flange there is zero. If we attempt to take a flange thickness at the height of point of contact of flange and rail, this, too, is not definite. The contact is not a point, but a line inclining away from the track and from the vertical, and not having a determinate distance. A graphical solution, made by shifting templates of treads of full size against full-size rail heads, is the only method that even approximates accuracy, and at the same time allows the mind to grasp the whole situation quickly and well, and to make proper comparisons. To attempt to define the situations by figures is like attempting to describe a picture or a scene by figures and dimensions.

The first difficulty met in an examination of this sort is to know just what the gauge of a track is. It seems strange that after so many years of railway experience this should not have been officially defined by any large body of officials. Many consider the gauge as between the upper corners, *E G*, Fig. 2, of rail heads. On one or two roads, and in at least one large manufacturing establishment for frogs and switches, the gauge is taken at mid-height of the rail head. On very few standard drawings is this point definitely determined. In this examination, the gauge of 4 ft. 8½ ins. has been taken as between the points *C D*, Figs. 2 to 5, because the shoulders of the track gauge rod are at right angles to the shaft, and *C* and *D* are the parts of the rail heads in contact with the shoulders, thereby determining the gauge. The flangeways, on the other hand, are often found to be measured with a foot-rule, and improperly between the points *E F* and *G H*, if the rail head be sloping. The proper plan is to measure with a guard rail gauge direct from the side of the frog point to the working side of the guard rail. These are the two points directly concerned in the guard-rail arrangements. The Master Car-Builders' Association formerly had a standard guard-rail gauge (see Fig. 11, Plate LXX). It was of poor design, because too much was attempted in it. It was

arranged, not only to give a distance from the side of the frog point to the working side of the guard rail, but it was also supposed to give the distance from the wing rail of the frog to the guard rail, and from the wing rail to each main rail. At the last meeting of the Association, at Lakewood, it was abolished as a standard, perhaps partly because of the foregoing reason, but also because it was considered that such a gauge was not within the province of that Association. It is properly a matter for the American Society of Civil Engineers, acting in concert with the Roadmasters' Association.

The distance from the side of the frog point to the working side of the guard rail also seems to be indefinite on many standard plans of railway companies. One of the most important roads in this country issues a standard plan with no widths of flangeways stated. The distance from wing rail to guard rail is given at 4 ft. 5 ins. between working sides, but it is not stated whether this is at top or bottom of the rail head. Some of the frogs of that company have a flangeway of 2 ins., leaving $1\frac{1}{2}$ ins. for guard-rail flangeway, with track gauge 4 ft. $8\frac{1}{2}$ ins.

All the examinations, Figs. 2 to 5, are with the 1885 or least objectionable set of wheel gauges. It was considered that whatever was found objectionable for this system would, of course, be more objectionable for the 1887 system; and that it was necessary to test the latter only upon an arrangement satisfactory for the 1885 gauges, namely upon Fig. 5.

Figs. 2 to 5 are all for a gauge of 4 ft. $8\frac{1}{2}$ ins. with unworn flanges. A worn flange lies further away from the frog point, and does not require consideration in this connection as regards spacing. Various locations of the guard rail with reference to the frog point have been taken, and the interference of a maximum gauge with the frog point is shown in these figures. Any guard rail distance $K\ C$, less than 4 ft. $6\frac{7}{8}$ ins., or guard rail flangeway $K\ D$, greater than $1\frac{5}{8}$ ins., with track gauge as noted, will give an interference at the frog point. A distance somewhat greater than 4 ft. $6\frac{7}{8}$ ins. might be better.

Placing the 1887 wheel gauges on such an arrangement as that shown in Fig. 5, we see in Fig. 6 (Plate LXIX) that the 1887 maximum wheel gauge gives a large interference at the frog point. If we allow for wear of the guard rail, the interference is, of course, increased. Where guard-rail flangeways of 2 ins., or even more, are used, it will

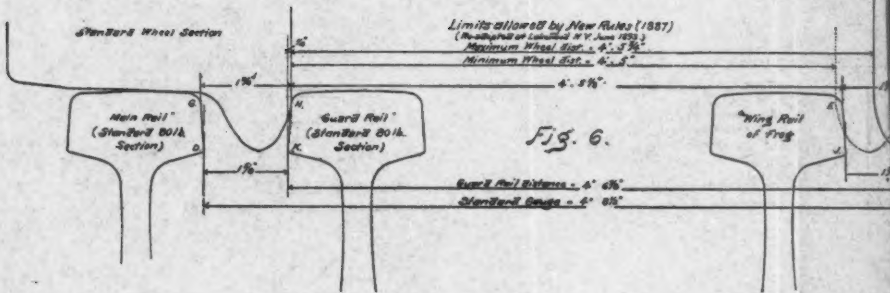


Fig. 6.

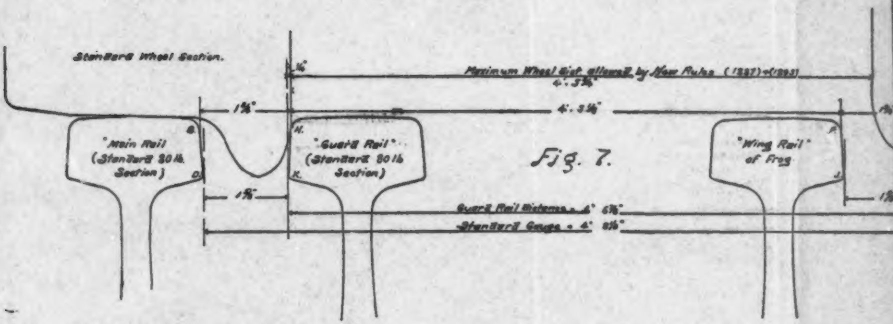


Fig. 7.

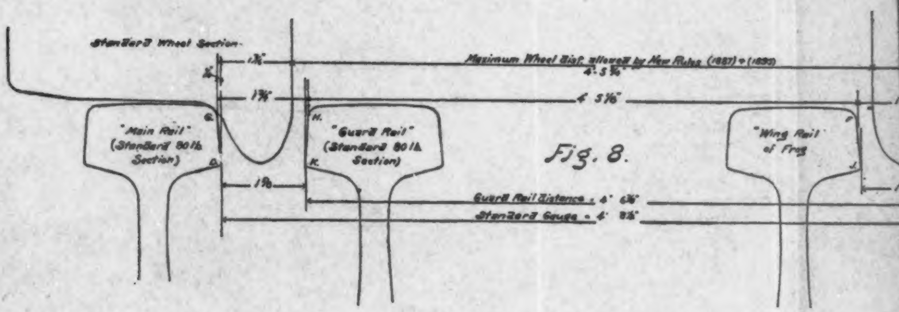


Fig. 8.

PLATE LXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.

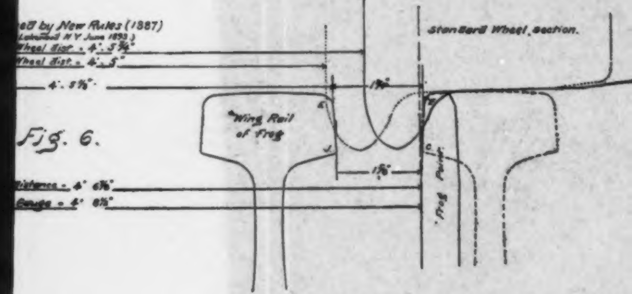


Fig. 6.

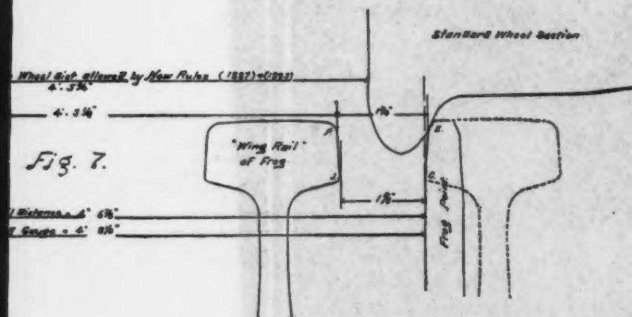


Fig. 7.

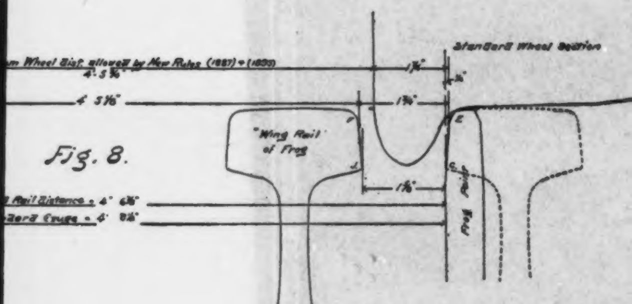


Fig. 8.

POINTS.

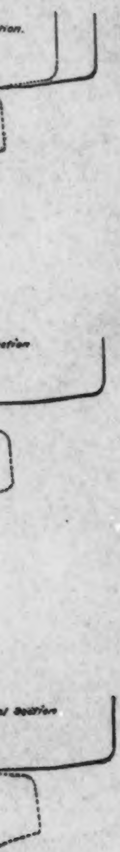
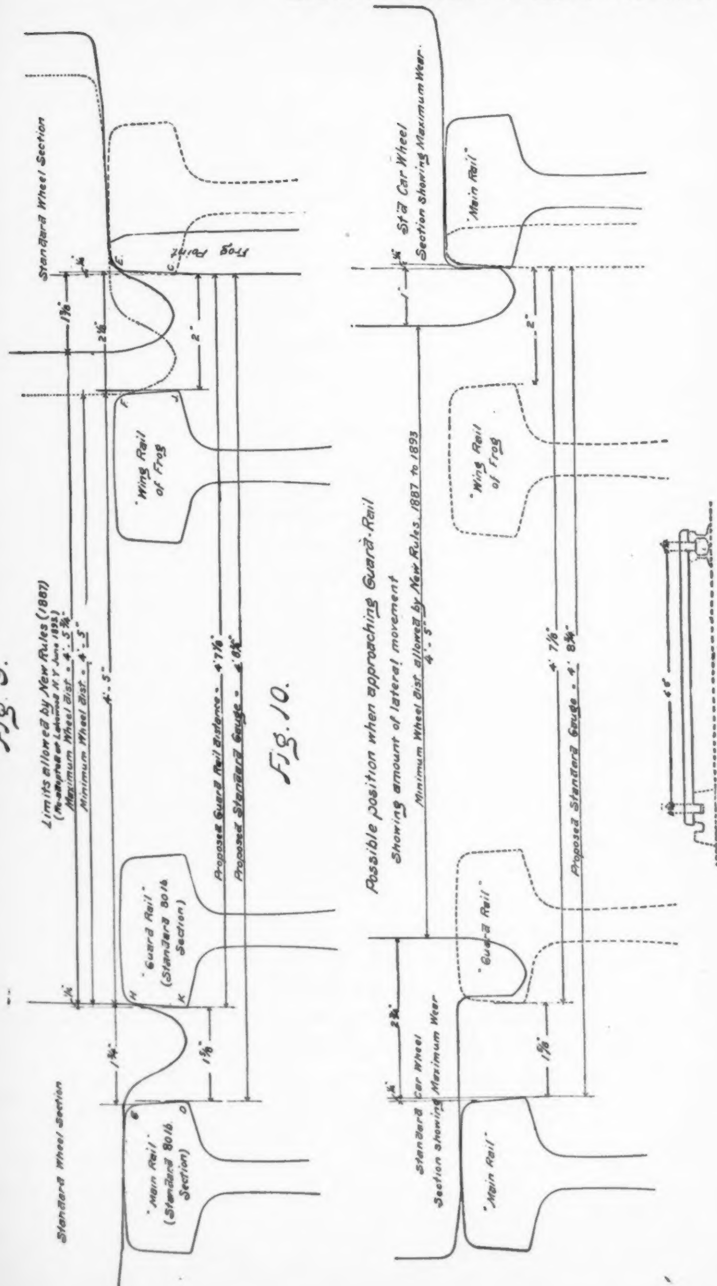


PLATE LXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.



be seen that the interference amounts almost to putting the flange on the wrong side of the point. Probably nothing but the impetus of the train carries a wheel of maximum wheel gauge safely past the frog point, in effect there being no guard rail. The 1887 minimum wheel gauge gives some interference with the wing rail. Fig. 7 (Plate LXIX) shows the first movement that probably takes place with the large maximum, running towards the frog point, with the arrangement of rails the same as in Fig. 5. The wheel momentarily rides on its flange and rises, the tread having no bearing. This gives a heavy blow and lateral pressure on the frog point. The second movement is probably that shown in Fig. 8, a shifting of the wheels with a heavy lurch, the guard rail becoming inoperative.

Fig. 10 (Plate LXX) shows how large the lateral movement becomes as the guard rail is approached, with the 1887 minimum and a worn flange. For this reason the track gauge should not be widened more than is necessary in any case. It also gives an additional reason for a return to the 1885 wheel gauges; and for the use of a long taper to the guard rail.

To accommodate the 1887 gauges while they remain in force, both the distance from the frog point to the guard rail, and the width of flangeway in the frog, require enlargement. Fig. 9 (Plate LXX) shows an arrangement that may be fairly satisfactory to those who do not demand a narrow flangeway in the frog. The track gauge is enlarged to 4 ft. $8\frac{3}{4}$ ins. But the pressure from roads of 4 ft.-$8\frac{1}{2}$ in. gauge has of late been so great to reduce flangeways in frogs to $1\frac{3}{4}$ ins., that the arrangement of 2-in. flangeway in Fig. 9 will not be generally acceptable. There is no alternative but a return to the 1885 wheel gauges.

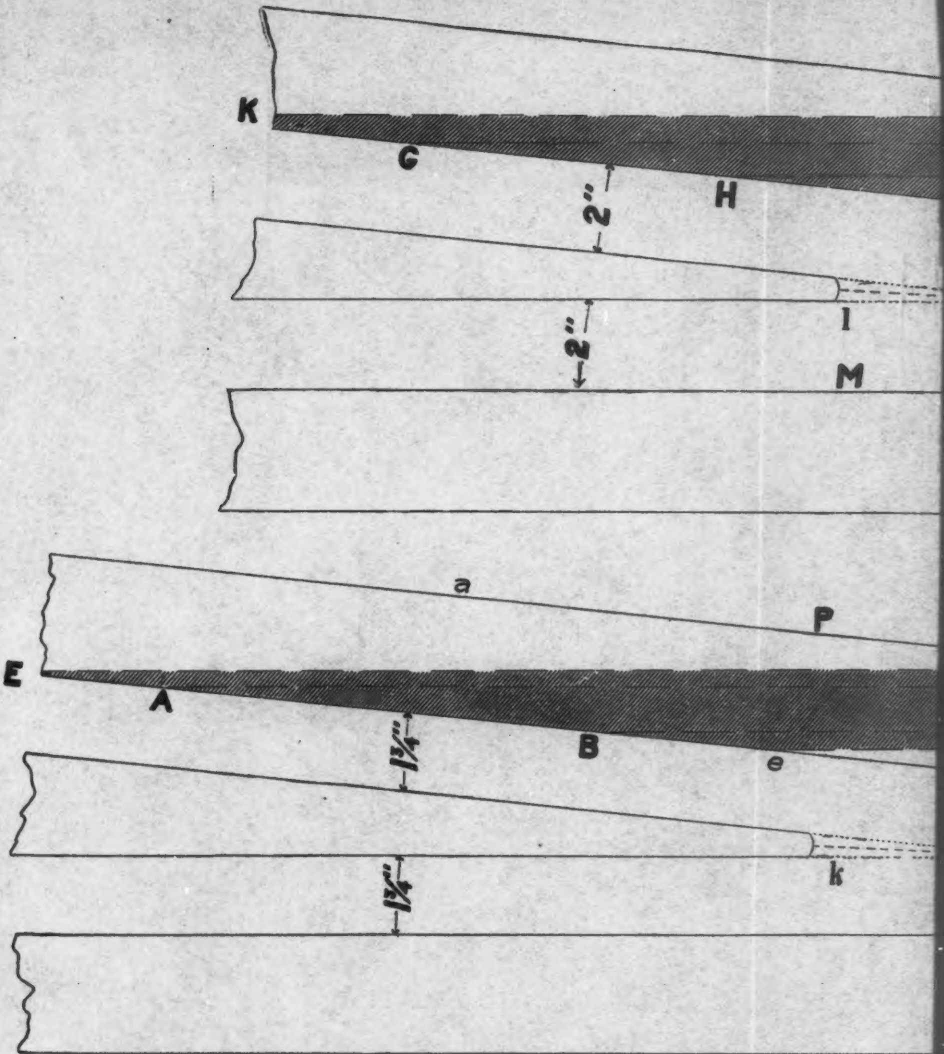
Although these broad flangeways of 2 ins. are used in many parts of this country (chiefly on roads of 4 ft. 9 ins. or compromise gauge) most of the roads using such a flangeway would probably prefer to reduce it to $1\frac{3}{4}$ ins. The table on page 514 shows the flangeways in use on many roads. The figures were not obtained direct from these roads in most cases, but from parties making frogs for them, and should be reliable. Where two widths are given, they are as given by two frog companies.

Engineers, roadmasters and frog-builders do not seem to be of one mind as to why frog flangeways should be reduced to $1\frac{3}{4}$ ins. or less.

STATEMENT OF WIDTHS OF FLANGEWAYS USED IN FROGS.

NAME OF ROAD OR MANUFACTURING COMPANY.	FLANGEWAYS USED IN RIGID FROGS.	
	Inches.	
B. C. R. & N. Railway Co.	1⅝	Measured at bottom of rail head.
N. Y. C. & H. R. R.R.	1¾	“ “ “
L. S. & M. S.	1¾	
N. Y. C. & St. L.	1¾	
Baltimore & Ohio	1¾	
“ “	2	At bottom of head.
T. & O. C.	1¾	
C. L. & W.	1¾	
C. & W. Indiana R. R.	1¾	At bottom of head.
Northern Pacific R. R.	1¾	“ “
Wisconsin Central Co.	1¾	“ “
C. M. & St. P. R. R.	1¾	“ “
C. St. P. M. & O. R. R.	1¾	“ “
M. & St. L. R. R.	1¾	“ “
J. & S. E.	1¾	“ “
M. K. & T. Ry. Co. Partly	1¾	“ “
“ “ “ “	1⅞	“ “
T. A. A. & N. M. Ry. Co. “	1¾	“ “
“ “ “ “ “	1⅞	“ “
C. & G. T. Ry. Co. “	1¾	“ “
“ “ “ “	1⅞	“ “
St. P. Union Depot Co. “	1¾	“ “
“ “ “ “	1⅞	“ “
M. St. P. & S. St. M. R. R. “	1¾	“ “
“ “ “ “	1⅞	“ “
A. T. & S. F. R. R. “	1¾	“ “
“ “ “ “	1⅞	“ “
St. Paul & Duluth R. R. “	1¾	“ “
“ “ “ “	1⅞	“ “
Cleveland F. & S. Co.	1¾	For 4 ft.-8½ in. gauge.
“ “ “	2	For 4 ft.-9 in. gauge.
C. H. V. & T.	1¾	At bottom of head.
“ “	2	
Penna. R. R.	2	
Michigan Central	2	
C. & Northern Pacific R. R.	2	
Wil. & Northern	2	
Cumberland Valley	2	
Western Maryland	2	
W. N. Y. & P.	2	
C. F. & Y. V.	2	
Ohio & Mississippi	2	
Chicago & Erie	2	
C. S. & H.	2	
Phila. & Reading	2	
C. H. & D.	2	At bottom of head.
T. St. L. & K. C.	2	
Pittsburgh & Lake Erie	2	Meas. ⅝ in. below top of rail head.
N. Y. L. E. & W.	2	At bottom of head.
Wheeling & Lake Erie	2	“ “
Pan Handle R. R.	2	At middle of head.
T. St. L. & K. C.	2	Main track flangeway gauge 4 ft. 8½ ins.
“ “	1½	Side track flangeway gauge 4 ft. 8½ ins.
Chicago & North Western	1⅞	At bottom of head.
C. C. C. & St. L.	2⅛	“ “
New England Roadmasters' Ass., 1884	1¾	Will reduce to less than 1¾ ins. when gauges become more uniform.
Indianapolis Switch & F. Co.	2	
Ramapo Iron Works	1¾	For 4 ft.-9 in. gauge.
“ “ “	2	For 4 ft.-9 in. gauge.
Carlisle Manufacturing Co.	2	Meas. at mid height of rail head.
Union Switch & Signal Co.	2	Usual width in all frogs made by them. Measured at ⅝ in. below top of rail head.
Pennsylvania Steel Co.	2	
“ “ “	1¾	Preferred when wheel and track gauges are reduced.

NOTE.—In most cases these measurements are taken at bottom of rail head.


K
G
H
2"
2"
l
M
a
P
E
A
B
e
1 3/4"
1 3/4"
k
b
O

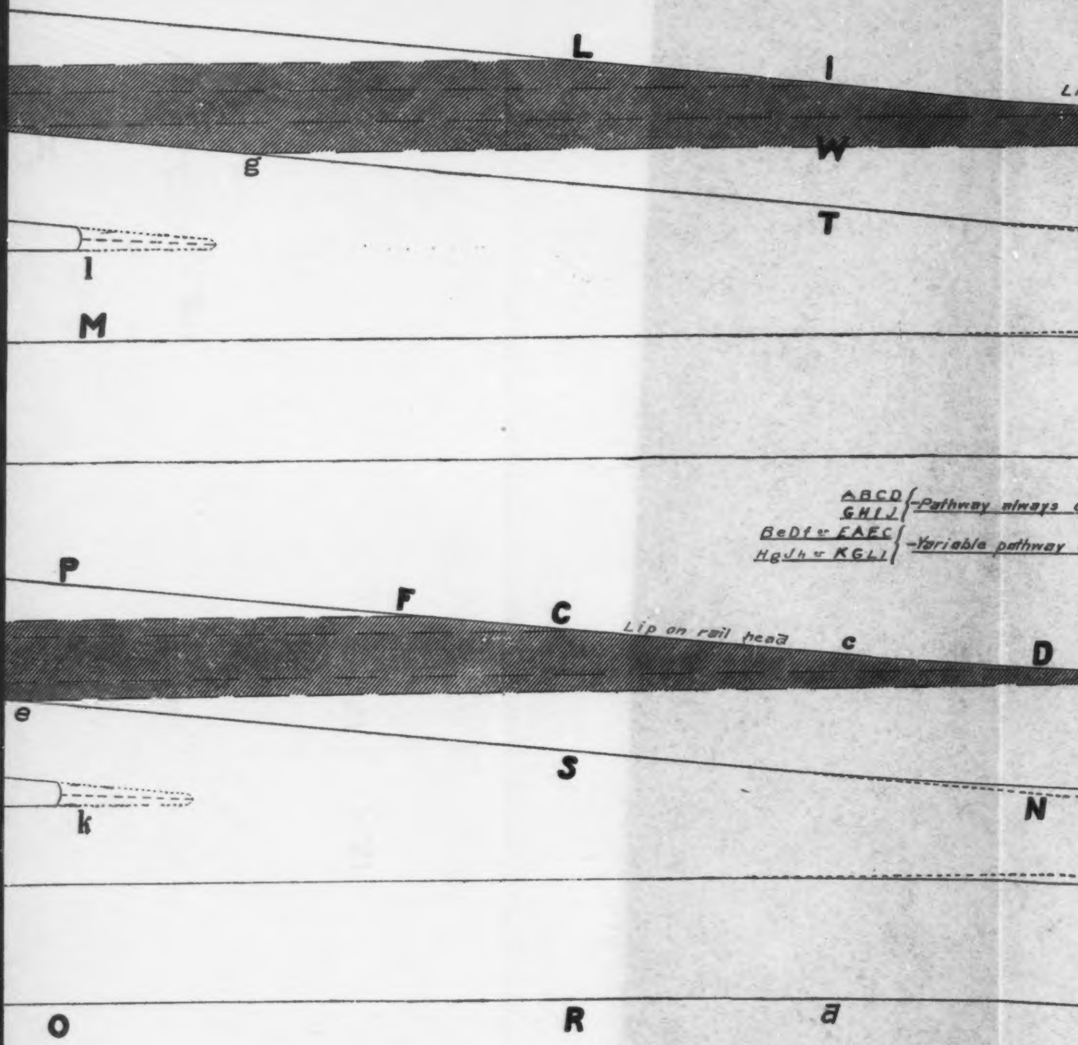
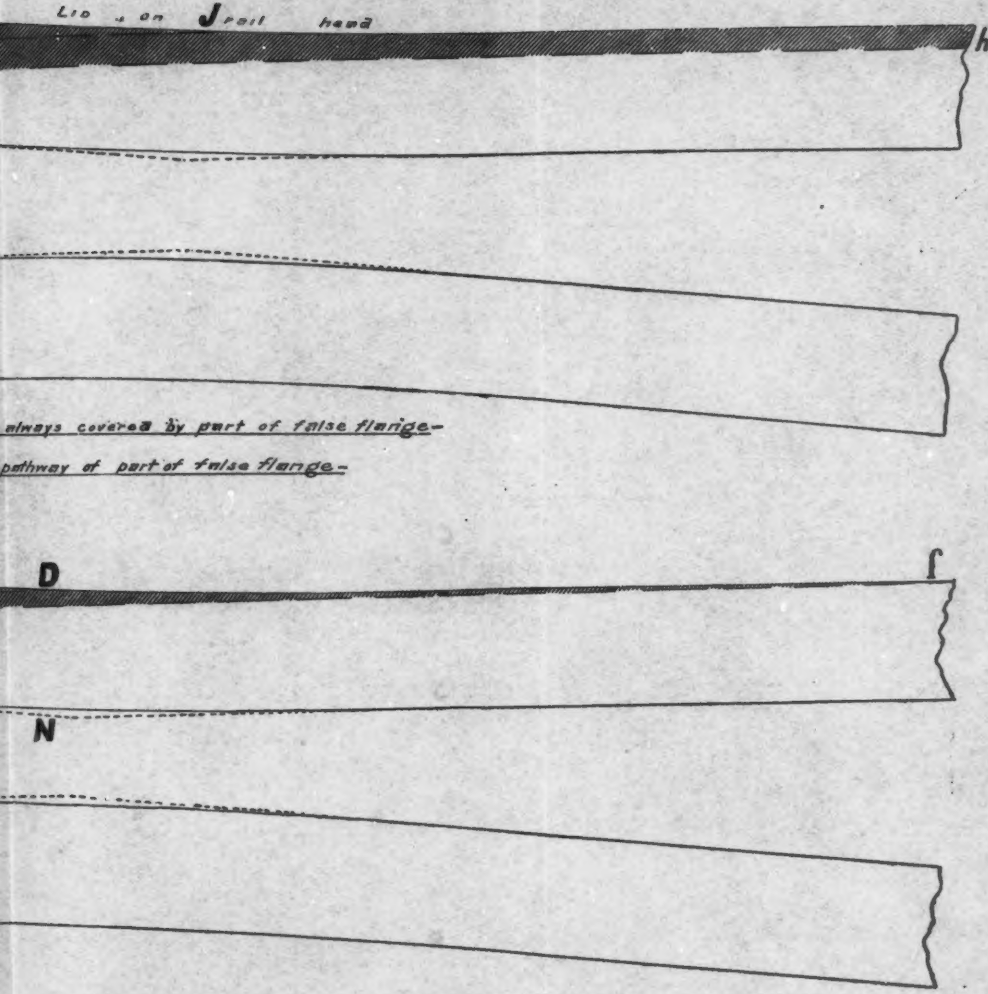

L
I
W
g
T
l
M
ABCD
GHIJ
-Pathway always
BeDf or EAEC
HgJh or KGLI
-Variable pathway
P
F
C
Lip on rail head
c
D
e
S
k
N
O
R
a

Fig. 12.

PLATE LXXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.



Some of the reasons, as noted in operating or claimed to exist, are as follows:

First, and perhaps the most important.—The rules of the Master Car-Builders' Association allow a loss of $\frac{3}{8}$ in. from rim of tread. This portion constitutes the chief part of the wheel that is to carry the wheel past the throat of the frog, as at *M*, Fig. 12 (Plate LXXI). See also Fig. 16 (Plate LXXIII). The Master Car-Builders appear to have overlooked the value of the outer rim when at the frog point. This reduction, combined with the breakage of a little more when the wheel is at the throat of the frog, lets the wheel down into the throat, causing derailment. Hence the narrower the throat is, the better; and a return to the 1885 wheel gauges is called for. In this connection it would be well to give attention to the filler bearing, which is the secondary bearing in cases of chipped rims.

Second.—With the broad flangeway, the coning of the outer part of the tread allows the wheel to drop deeper into the throat of the frog. With new wheels this causes a blow in running facing point, or a sudden drop to the wing rail bearing, in running trail (see Fig. 16).

Third.—The wear on wing rails is claimed to be greater with the broad flangeway. At first sight this would appear to be the case. An increase of $\frac{1}{4}$ in. in each flangeway, giving apparently $\frac{1}{2}$ in. less bearing of tread on the wing rail, would appear to increase the wear and destruction of the wing rail. In reality there is no gain of increased bearing on the wing rail from the use of narrow flangeways. There is a slight gain in the following respect, namely, that the point where the tread leaves the wing rail is shifted further from the frog point, in a narrow flangeway. With a 2-in. flangeway, the tread leaves the wing rail at *G* (Fig. 12). With $1\frac{3}{4}$-in. flangeway, this point is shifted to *A*, or, in a No. 10 frog, about 5 ins., giving a little wider bearing on the frog point, although the increase is not great, nor as much needed as in the case of a rim bearing. So far as an increase of rim bearing is concerned, it will be noted that the outer part of the tread is crossing the wing rail at an angle. Whatever the flangeway width, there must be a triangular area, *S A C* or *T G I*, where the width of bearing is gradually lessening. With either of the flangeway widths named this point *A* or *G* is far enough from the frog point for the point to furnish a considerable width of bearing. It is therefore doubtful whether narrowness of tread bearing is a large factor in frog

wear. This is still more evident if we consider the effects of false flanges, which is the agency most destructive in frogs. Fig. 12 is intended to illustrate such wear from false flanges. This wear will be found most largely in yards, where the treads of shifting engines are for some reason allowed to have very deep hollows, although running over frogs constantly. The maximum hollow of tire allowed is assumed to be about $\frac{3}{8}$ in. in shifting engines, and about $\frac{3}{16}$ in. in wheels in general service. The false flange simply crosses the wing rail from one side to the other, as indicated by the shaded strips *A B C D* or *G H I J* in Fig. 12. In many frogs in yard service this line will be found either somewhat grooved, if the frog be old; or merely polished, if the frog be new, while much of the area *D E N* or *W G T* is often dull or sometimes almost rusty. The strips *A B C D* or *G H I J* become channeled out soon. These channels or grooves are much deeper at each end than in the central portion of their length. A lip is formed on the rail outside; and would be formed also where the wheel leaves the wing rail inside, but for the wear of flanges against it. A lip is also formed on top of the rail, outside of the line of false flange-bearing. When the areas *D E N* or *W G T* become much higher than the channels cut by the false flanges, these areas are worn down rapidly by new wheels, and in general do not long remain much higher than the false flange grooves. This wear from false flanges is so great in many situations as to be the only one worthy of consideration, and in such cases flangeway widths are not of importance, so far as wear is concerned. Figs. 13, 14 and 15 (Plate LXXII) show in section approximately some of the effects of false flanges. Engine treads widened to 5$\frac{3}{4}$ ins. have an injurious action where false flanges of treads of the ordinary 5$\frac{1}{2}$ ins. have worn a groove. The outer portion of such grooves being curved, concave upwards, the added $\frac{1}{4}$ in. of tread rests on this curved surface, giving an outward strain on the frog, and probably affecting the axle.

In examining to what extent the narrow flangeway is more desirable than the broad flangeway, many other considerations than mere width of tread bearing should be kept in mind. One consideration is that simultaneous bearing on both frog point and wing rail are seldom secured. Usually either the point alone or the wing rail alone carries all the weight. When both point and tread are new, the bevel on the outer edge of the tread throws all the bearing on the frog point (see Fig.

PLATE LXXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.

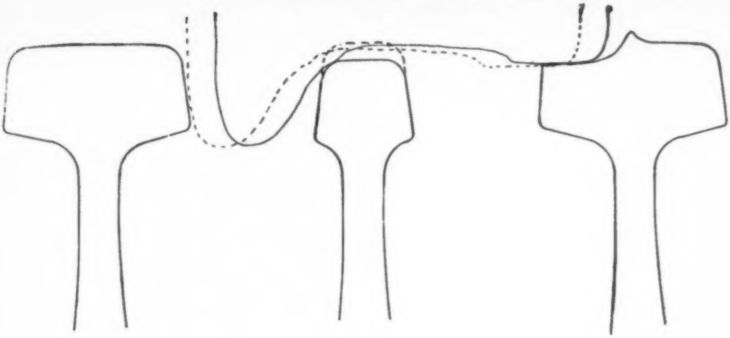


Fig. 15
— Section at a. b —

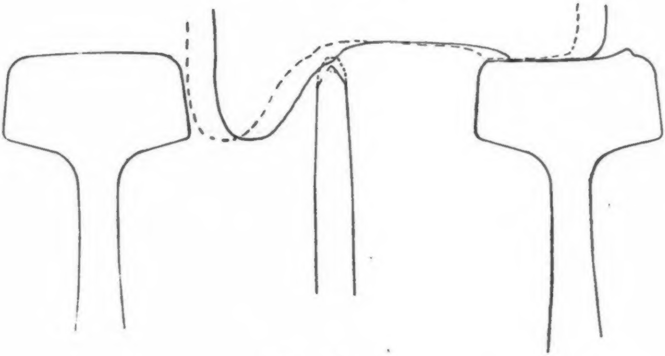


Fig 14
— Section at O. P. —

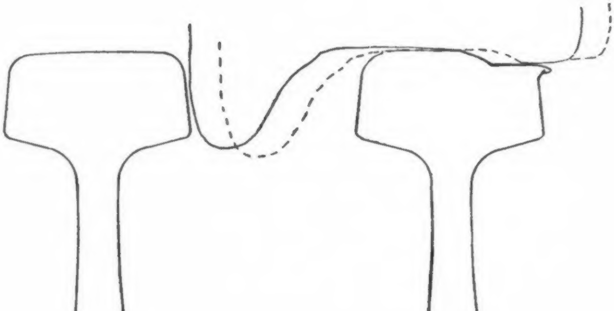


Fig 13
— Section at c. a. —

PLATE LXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.

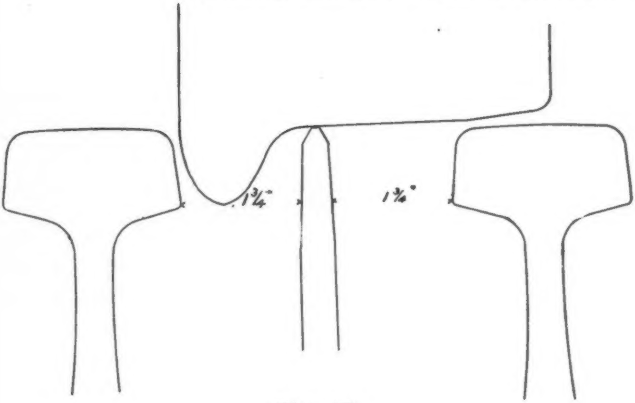


Fig 17
Section at O.P.
New wheel and rail

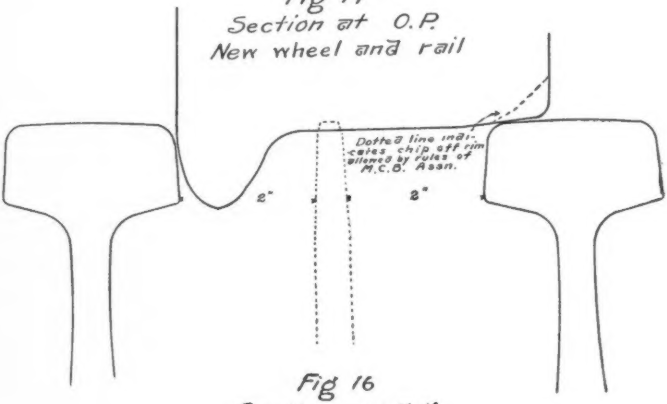


Fig 16
Section at H.M.
New wheel and rail

17, Plate LXIII). A wing rail badly worn by false flanges, or a yielding wing rail, has the same effect. A wing rail bent vertically, part of it low, part "cocked up," also throws all the bearing on the frog point in some cases. In all such cases, the width of flangeway is not of importance. On the other hand, many frog points will be found to "duck" as the wheel approaches them, leaving the wing rail to carry all the weight. The lack of base which the rail forming the frog point has at the point, tends to allow this "ducking." In this case, the width of tread bearing is important, to carry the wheel firmly until the depressed portion of the frog point has been passed. Hollow tires also prevent simultaneous bearing. These failures to secure simultaneous bearing make it desirable to ignore the frog point as a bearing for some distance from the point, and to make narrow flangeways and a return to the 1885 wheel gauges desirable. But so far as wear from broad flangeways is considered, this effect can readily be made too much of.

The destruction of a frog is generally primarily due to shock. This acts both by making the frog too loose and shaky for use, and by indirectly increasing the wear because of such a loose state of the frog. Hence, narrow flangeways will not make a very great increase in the life of a frog unless it is well cared for in other respects. The shock to a frog is generally assumed to be the blow upon the point. It is probable that the lateral blow at *A* (Fig. 12), and also in the angle where the point rails come together, from the false flange of a wheel running trail, strains the frog by a wedge-like action and loosens it more than a blow on the point does. The false flange, before climbing to the height of the rail head, must throw a severe strain on the frog. Such action will take place equally with a 2-in. or a 1¾-in. flangeway.

We may summarize as follows:

1. The necessity of protecting the frog point laterally calls urgently for a return to the 1885 wheel gauges.

2. The dangers from narrow or broken treads call for similar action, and for a modification of the rule about chipped rim of tread.

3. A definite rule about maximum false flange or hollow tire is called for.

4. A return to the 1885 wheel gauges would result in only a moderate gain in durability of frogs, so far as vertical action or wear by wheels is concerned.

5. The appliances used in some spring rail frogs to assist the false flange to climb to the level of the rail head would probably increase the durability of all frogs.

The Master Car-Builders' Association at its last meeting appointed a committee to "prepare maximum gauges for thickness of wheel flanges, and to consider wheel gauges in their relation to the track, and to offer any suggestions as to wheel and track gauges, and to confer with any similar committee from other associations, should such be appointed."

DISCUSSION.

M. J. Becker, Past Pres. Am. Soc. C. E. (by letter).—The "Relation of Wheels to Frog Points and to Guard Rails," which Mr. A. A. Schenck has chosen for the subject of his paper, is a topic which should receive very careful attention and earnest consideration by engineers charged with the construction and maintenance of our railways.

It is but a few years since the gauges of our railway tracks ranged all the way from 5 ft. in the South through the intermediate gauges of 4 ft. 8½ ins. in the West, and 4 ft. 10 ins. in the East, to the 6-ft. broad gauge of the Erie system, compelling, for the interchange of traffic, such makeshifts as double lines of rails, and hoisting machinery for the exchanges of trucks at the connecting points, and broad tread compromise wheels for all rolling stock engaged in through traffic. How the wheels during their periodical changes, and with the always liberal permissible variations between the maximum and minimum gauges, managed to pass through the various flangeways of the switches and frogs seems more of a puzzle to-day than it appeared at the time when this state of affairs existed.

At last the track gauge question is practically settled by the almost universal adoption of the 4 ft.-8½ in. gauge. Only the lines of the Pennsylvania system and a few of its connections still adhere to the 4 ft.-9 in. gauge, which was adopted about 15 years ago as a compromise between the 4 ft.-10 in. and 4 ft.-8½ in. gauges.

The play between the present wheel gauge and the 4 ft.-9 in. track gauge on these lines is 1¼ ins. when rail heads and wheel flanges are new and perfect, and is likely to be more when the rail heads and wheel flanges are reduced by wear. This play causes undue lateral motion and severe shocks at the frog points, and the question of reducing the gauge to the 4 ft.-8½ in. standard is now under consideration.

It was during the investigation of the effect which would be produced by existing rolling stock with its permissible variation of gauge upon the reduced track gauge, with its accompanying frog throats and guard-rail flangeways, that I found the condition shown in the following table:

Case.	Track gauge.	Guard-rail gauge.	Gauge of wheel tread.	Gauge between wheel backs.	Flangeways of frogs and guard rails.	Play between wheel back and guard rail.	Play between track gauge and wheel-tread gauge.	
1.....	4′ 9″	4′ 5″	4′ 7¾″	4′ 5″	2″	None.	1¼″	Close fit.
2.....	4′ 9″	4′ 5″	4′ 8½″	4′ 5¾″	2″	¾″	½″	O. K.
3.....	4′ 9″	4′ 5½″	4′ 7¾″	4′ 5″	1¾″	¼″ less than clearance.	1¼″	Impracticable.
4.....	4′ 9″	4′ 5½″	4′ 8½″	4′ 5¾″	1¾″	¼″	½″	O. K.
5.....	4′ 8½″	4′ 4½″	4′ 7¾″	4′ 5″	2″	½″	¾″	O. K.
6.....	4′ 8½″	4′ 4½″	4′ 8½″	4′ 5¾″	2″	1¼″	None.	Close fit.
7.....	4′ 8½″	4′ 5″	4′ 7¾″	4′ 5″	1¾″	None.	¾″	Close fit.
8.....	4′ 8½″	4′ 5″	4′ 8½″	4′ 5¾″	1¾″	¾″	None.	Close fit.

NOTE.—The thickness of wheel flange is taken as 1⅜ ins.; so that if the gauge between wheel backs is at its minimum of 4 ft. 5 ins., the gauge of wheel treads, which is the contact line between wheel flange and rail head, would be 4 ft. 7¾ ins. If the gauge between wheel backs is spread to 4 ft. 5¾ ins., the gauge of wheel treads would be 4 ft. 8½ ins.

From the above table it would appear—

First.—That with a track gauge of 4 ft. 9 ins. and flangeways of either 1¾ ins. or 2 ins., wheels with gauge of the extreme permissible limit of 4 ft. 5¾ ins. will pass freely and safely (Cases Nos. 2 and 4 in table above).

Second.—That with a 4 ft.-9 in. track gauge and 2-in. flangeways, wheels with the minimum gauge of 4 ft. 5 ins. will just pass, but rub hard with their backs against the guard rails and wing rails of frogs (Case No. 1 in table above).

Third.—That with a 4 ft.-9 in. track gauge and 1¾-in. flangeways, wheels with minimum gauge of 4 ft. 5 ins. will lack ¼ in. for clearance of the guard rails and wing rails of frogs, and will not pass with safety (Case No. 3 in table above).

Fourth.—That with a 4 ft.-8½ in. track gauge, wheels with gauges of the maximum permissible limit of 4 ft. 5¾ ins. will pass freely through flangeways of either 1¾ ins. or 2 ins., but the wheel flanges will rub very hard against the head of the main rail (Cases Nos. 6 and 8 in table above).

Fifth.—That with a 4 ft.-8½ in. track gauge and 2-in. flangeways,

wheels with the minimum gauge of 4 ft. 5 ins. will pass freely and safely (Case No. 5 in table above).

Sixth.—That with a 4 ft.-8½ in. track gauge and 1¾-in. flangeways, wheels with the minimum gauge of 4 ft. 5 ins. will just pass, but rub hard with their backs against the guard rails and wing rails of frogs (Case No. 7 in table above).

This shows that with a 4 ft.-8½ in. track gauge, flangeways of 2 ins. width are safer than those of 1¾ ins. width as long as the existing wheel gauges remain in use, and that a judicious widening of the track gauge on curves is necessary.

It also shows that with a 4 ft.-9 in. track gauge the inadvertent purchase and use of a frog with a flangeway of 1¾ ins., of which quite a number are made throughout the country for lines with 4 ft.-8½ in. gauges, would in all probability have caused a wreck, even if the flangeway at the guard rail had been 2 ins.

By the adoption of spring rail frogs, with beveled outside rail attachments for carrying the worn wheel treads over the frog points, some of the difficulties pointed out by Mr. Schenck can be overcome, but I quite agree with him that a determined effort should be made towards the adoption of a standard method for determining the points on the rail heads and frog points at which the track gauge and guard-rail gauge should be applied, and that the question of modifying the permissible variation in the wheel gauge should be considered by the Master Car-Builders' Association in conjunction with the engineers in charge of the tracks.

HENRY G. PROUT, M. Am. Soc. C. E.—I have not had time to read this paper carefully enough to make any extended discussion of it. I will merely suggest that the question is one which requires co-operation on the part of the people in charge of track work with the people in charge of the rolling stock. It is a fact that there is a very great variation, not recommended by any of the standards, but entirely arbitrary, in the thickness of wheel flanges. Nominally the working thickness of wheel flanges between the point where the wheel comes in contact with the rail head and the back of the wheel is 1¼ ins.; actually it very often reaches 1½ ins. and sometimes 1¾ ins. This variation is enough to cause the wheel to strike against the point of the frog, when if the theoretical gauge—the Master Car-Builders' gauge—was used, it would not do so. The Master Car-Builders' rules of interchange accept a car wheel which enters the minimum gauge, but they have no maximum gauge for the thickness of the wheel flange. The consequence is that one runs into the danger of having the wheel flange so thick that the prescribed standards will actually permit a wheel that will strike the point of the frog.

GEORGE R. HARDY, M. Am. Soc. C. E.—The question of the relation of the track gauge and wheel gauge is a very interesting one, and

it brings to mind some very peculiar points. The track gauge undoubtedly is a distance which can be definitely determined. The width usually adopted for the track gauge can be measured between vertical planes, and with rails not formed with a very excessive inclination on the sides of the heads the gauge is very well determined. Different forms of rail head only slightly complicate the exact determination. If a railroad company adopts a form of head with considerable inclination to its sides, then the gauge becomes wider than on a road where the standard rail has vertical sides. It is probable that few realize how these railroads go on operating year after year, with only this little flange of the wheel to keep the track, running at such high speeds, 30 miles or more per hour, and when we consider that there is only about $1\frac{1}{8}$ ins. of wheel flange to protect the whole train service, it is one of the most wonderful things in the world.

I think that we may correctly affirm, notwithstanding the variation in rail heads, that the gauge is accurately defined; but when we come to the rolling stock it is impossible for the Master Car-Builders and superintendents of motive power to determine what that gauge is; it varies according to the relative positions of the wheel and the rail. The problem which was encountered in trying to define an actual gauge for the wheel remains unsolved, and as a temporary expedient or makeshift the mechanics have tried to relieve themselves of the responsibility by prescribing how far apart the backs of the wheel should be. That is much easier because the backs of the wheel can be limited by verticals which may be compared with the distance between the verticals of the rails which measure the gauge, and the difference thus obtained is made up of the slackness plus twice the thickness of the flange. The different positions in which the wheel may be, with reference to the rail, makes a difference in the bearing points of the curves of the wheel and the rail head. If these two curves were alike, the problem might be solved. Four or five years ago the question was in discussion, and at that time it was suggested that a possible actual comparison could be made, if we would make the curve of the rail head and the curve on the wheel that bears against it of the same radius; then, by determining the distances between the centers of each set of two curves, the actual slack of the two gauges could be determined, and also the flange thickness. I do not know that any advance has been made in that direction. It may be that the mathematical determination of what the difference is between the rail gauge and the wheel gauge is unimportant; but in any case the track gauge is defined, while the responsibility of defining a wheel gauge remains unsatisfied. Engineers have tried to determine just how much slack they should give the gauges of the track for curvature. With $\frac{3}{8}$ in. or $\frac{1}{2}$ in. slack in the difference between the two gauges, it seems absurd to think about the question of widening the gauge on curves until we get up to very sharp curves.

H. W. BRINCKERHOFF, M. Am. Soc. C. E.—I would like to mention a mechanical method of investigating a somewhat similar problem. The Johnson Company, of Johnstown, Pa., made a number of careful experiments to determine the proper sections for guard rails for street railways.

First, an ordinary street guard rail was bent to a given radius; then two wooden discs were made to represent the wheel flanges; these were fastened to a piece of wood so as to keep them in the same plane and the proper distance apart. Then the groove of the guard rail was filled with plaster of paris, and, before it had a chance to set, the wooden discs, connected as above, were slid along the rail, scraping out all the plaster that was in their way and leaving just the shape of the groove that they needed for their free passage and proper guidance. I may add that new guard rails, which promise greater durability and smoothness of running, have been designed in accordance with the result thus found.

It is very obvious that a somewhat similar method could be pursued in investigating the positions taken by a wheel in passing through a frog. If the frog were filled with plaster of paris and the wheel rolled through, it would leave its own record of the positions it had occupied in passing. That would seem to be a convenient way of investigating the question.

L. L. BUCK. M. Am. Soc. C. E.—With a larger wheel, it would take more plaster out of the groove of the guard rail. A larger wheel, or shorter radius, or longer wheel base, would each require a wider groove in the guard rail.

A. A. SCHENCK, M. Am. Soc. C. E.—I wish to allude briefly to one or two items that have been referred to in this discussion. I noticed, however, that the Secretary, in reading one paper, omitted nearly all the figures in it, thus justifying the impression my paper is intended to give, that examinations of this problem by figures is much less satisfactory than a graphical examination.

The information given by Colonel Prout about excess of thickness of wheel flanges has an important bearing on the problem. This excess of thickness, when not in combination with a maximum wheel gauge, is not necessarily a source of danger. The original amount of flange thickness is still left on the right side of the frog point. A flange of ordinary thickness put over a half inch too far upon the frog point might be more dangerous than a thick one, as less flange thickness would be left on the right side of the wheel, to keep the wheel in proper place. But there is an enormous increase of wear by a flange of excessive thickness in combination with the maximum wheel gauge now allowed. What actually occurs with the present maximum wheel gauge, is that the rolling stock is obliged to do on the road what the shopman and the trackmaster should do in advance, and make frogs

and gauges that are suited to the wheels. The wheels plough through the frogs much as a wagon wheel makes its rut in the country road; but by the time the operation is completed there is little left in the frog.

I see that Mr. Becker assumes the flange of wheel as 1⅜ ins. thick. He also advocates a guard-rail flangeway of 2 ins., and considers the maximum wheel gauge of 4 ft. 5¾ ins. admissible. A little blackboard exercise will show:

4 ft. 8½ ins.		4 ft. 5¾ ins.
2 "		1⅜ "
4 ft. 6½ ins.,	in which to put	4 ft. 7⅛ ins.

All of the many examinations made of this problem by figures have been based on assumptions of flange thicknesses that are purely theoretical. This flange thickness of 1⅜ ins. has no definite relation either to the track gauge or to the points of contact and of binding in actual practice. By taking the maximum wheel gauge of 4 ft. 5¾ ins. from 4 ft. 8½ ins., we have a balance large enough for two theoretical flange thicknesses of 1⅜ ins. By an "*argumentum in circulo*" we are then told that the maximum wheel gauge of 4 ft. 5¾ ins. plus two wheel flanges of 1⅜ ins. will go through the gauge of 4 ft. 8½ ins., although it will be a tight fit.

If we would get the normal closest position of a wheel against a rail head, we should shift the one against the other as closely as can be done without lifting the wheel tread off the rail head. Anything closer than this means that we are lifting the rolling stock off the wheel treads and carrying it on the curved flange of wheel. But a determining of a proper clearance of frog point or of a proper wheel gauge by the assumption of any one flange thickness is manifestly impossible, because of the curving outline of this wheel flange, as well as because of the curves in the rail head.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

707.

(Vol. XXXI.—May, 1894.)

THE USE OF CANVAS IN WATER-TIGHT BULKHEADS.

By M. MEIGS, M. Am. Soc. C. E.

READ MAY 2D, 1894.

WITH DISCUSSION.

The use of canvas stretched over loosely fitted boards, to form water-tight constructions, is not new, but is not as well understood by many engineers as it should be. It has a considerable value in engineering work, and, properly appreciated, can be used advantageously in many difficult situations which offer peculiarities requiring special treatment.

The special application which forms the basis of this paper will show one use to which canvas may be put, and possibly may suggest to some of our members experiences in the past or occasions in the future when canvas will solve a difficult problem in a neat and advantageous manner.

The writer considers the use of canvas specially adapted to the following cases:

First.—Construction of water-tight coffer-dams, especially on rock bottoms.

Second.—Suppression of leaks in embankments; as, for instance, the so-called sand-boils.

Third.—Protection of levees and other embankments at critical points during the invasions of high water.

Fourth.—Suppression of leaks in foundations, such as important reservoir dams. For raising sunken vessels, it has been long in use on the western waters of the United States, and probably in other countries.

The special instance referred to, and offered as an example, is the following:

The United States Mississippi River Canal, at Keokuk, Ia., in charge of Major A. Mackenzie, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and locally administered by the writer, has been in constant operation since 1877.

There are three locks, one at each end of the canal, and one about midway. The plan under which the canal was built provides no guard gates at either end, as is the general and very commendable practice at present. The two upper locks have been drained from time to time, and inspections and necessary cleanings and repairs made, as occasion demanded. The upper lock was on two occasions separated from the river by an ordinary coffer-dam of plank and mud, no gravel being at hand or easily gotten, and a dredge being available, which could furnish quickly any quantity of rather stiff and heavy silt or gumbo of the usual Mississippi River variety.

On each of these occasions the work had to be carried on after the close of navigation, under great difficulties from inclement weather, and on one of them the imperfectly consolidated dam, in a partly finished condition, upset just as a Dakota blizzard made its appearance and froze up towboat, dredge and four barges of mud, putting a stop to the work for the rest of that winter. This disagreeable experience was so well remembered by all connected with that episode that it was agreed that almost any other form of coffer-dam than a mud one was preferable for use after the close of navigation on the Des Moines Rapids Canal.

At the close of navigation, in 1893, November 15th, the lower lock had not been inspected for 16 years, and there was evidence in the working of the submerged machinery connected with it that the time had come when the lock must be drained, and examinations and proper repairs executed.

The filling and emptying culverts, 8 ft. wide and high, and the 16 iron culvert gates, 3 ft. wide and 6 ft. high, had to be examined, and the working of the latter showed that the slides on which they moved had to be replaced.

The leakage of the gates, too, had become serious, and their condition was a matter of speculation so far as the lower 9 or 10 ft. was concerned.

These gates are 46 ft. wide, each leaf, and about 27 ft. high, weighing some 40 tons each.

All these repairs and examinations were quite beyond the scope of a diver, and there was nothing to do but to drain the lock.

The canal being in constant use during the navigable season, the repairs had to be made in winter weather, which in these latitudes often sets in in the latter part of November.

The river is usually at a low stage in the latter part of November, but, as soon as the cold weather comes, the anchor ice causes sudden fluctuations in the level, generally rising to about a 4-ft. stage, until the river closes, when it may rise considerably farther, depending on the accidents of gorging below or above the foot of the canal. Hence a coffer-dam of less than 16 ft. high above the bottom of the river, or 8 ft. above low-water line, was not deemed safe. To have built this of gravel and timber in freezing weather was a job to be contemplated with little pleasure.

The writer had at his disposal a 50-H. P. suction dredge, with 14-in. suction, and a rotary Van Wie pump, also plenty of 12-in. discharge pipe, mounted on pontoons, and concluded to use this pump in preference to an engine on the lock wall, and an ordinary drainage pump, which would, however, have answered equally well, but was not on hand.

It was proposed to drain the lock with this dredge, allowing the boat to settle in the mud at the bottom of the lock as the water left it, and to complete the drainage with a 3-in. discharge pulsometer. The lock, being 350 ft. long and 80 ft. wide, a flat place on the bottom was selected and the dredge placed over it, the necessary length of discharge pipe placed in position on its pontoons, and this arrangement left nothing to be desired in the way of pumping machinery, as the result shows.

The point selected for the bulkhead and shown clearly in Fig. 1,

e 16
and
ved
ndi-
was
gh-
e of
the
des
per,
den
ntil
on
nce
or
s of
ted
-in.
rge
ref-
np,
on
the
it,
The
was
dis-
ent
the
1,

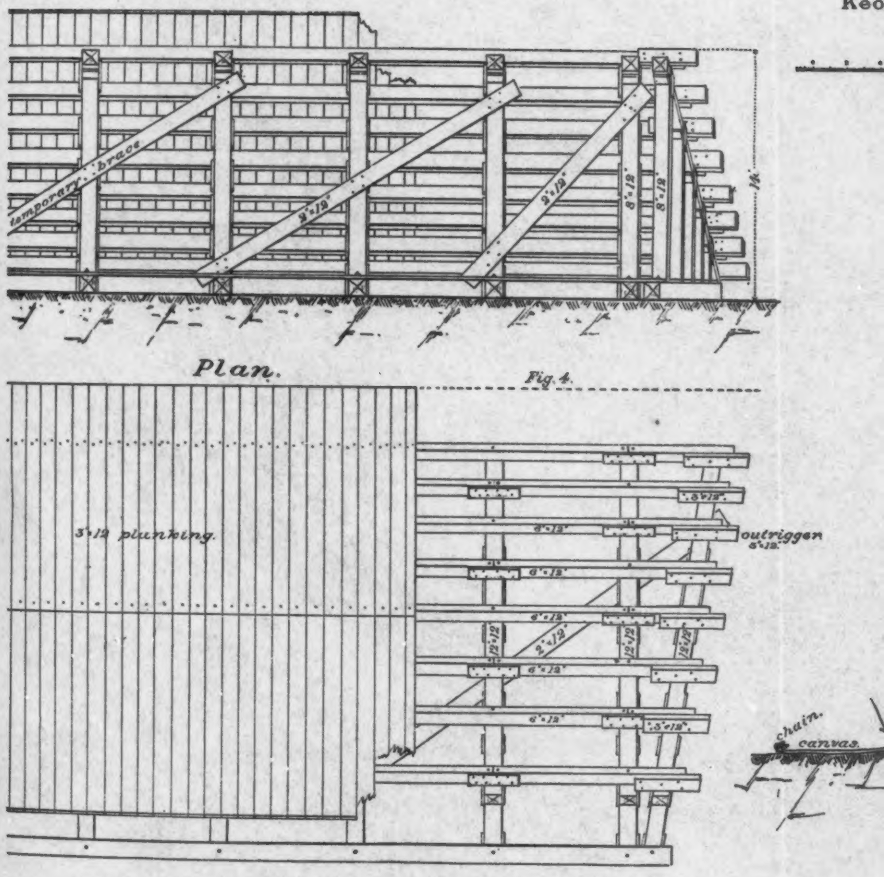
U.

Canvas a

a

Keo

Rear Elevation.



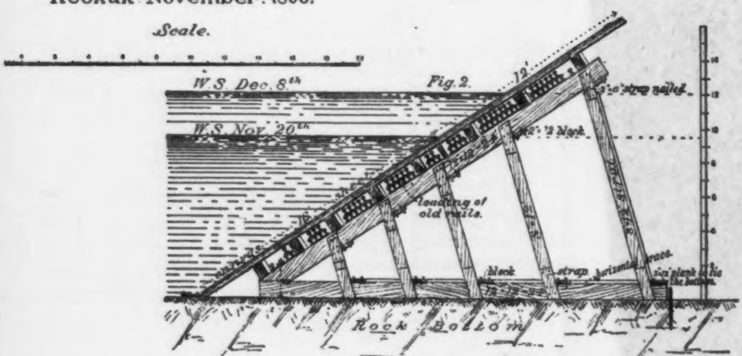
U.S.D.M.R.Canal.

anvas and Plank Bulkhead

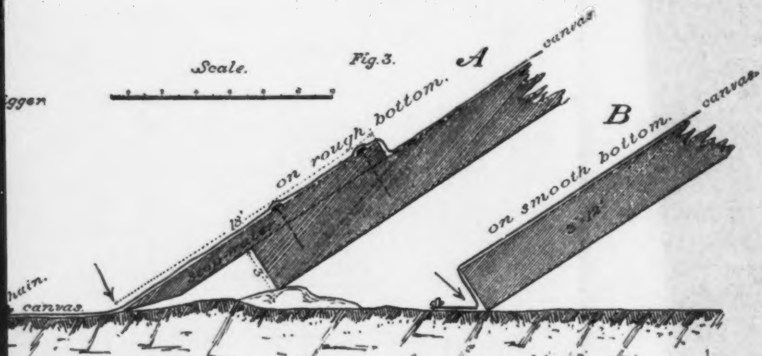
at Lower Lock.

Keokuk. November. 1893.

Scale.



Section 1.



T

MEI

iggen

hain.

PLATE LXXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 707.
MEIGS ON CANVAS FOR BULKHEADS.

Fig. 1.

GENERAL PLAN OF LOWER LOCK.

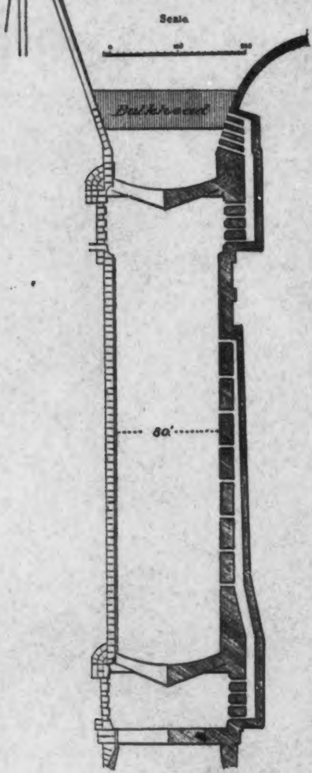


PLATE LXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 707.
MEIGS ON CANVAS FOR BULKHEADS.

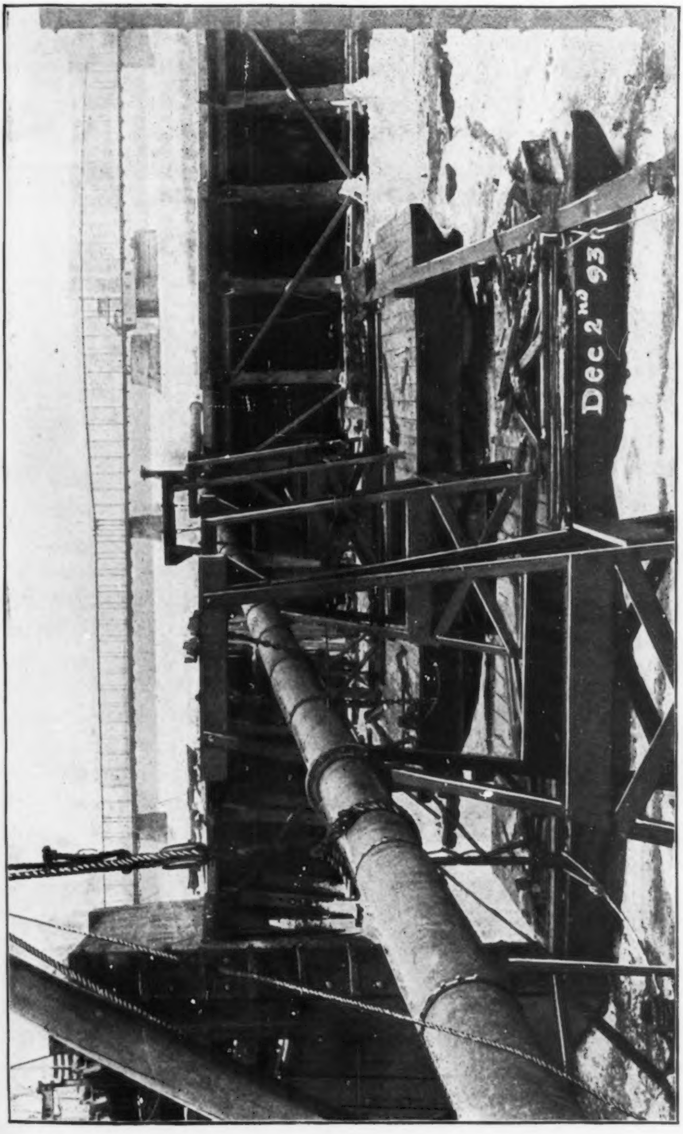


Plate LXXIV, was some distance (about 40 ft.) below the lower mitre-sill, where there was a smooth rock bottom, the ends of the dam abutting against the flaring ashlar wing walls of the lock approach.

The bulkhead consisted of thirteen triangular bents (Fig. 2, Plate LXXIV) of 12 × 12-in. timber for the rafter (or cap) and sill, and 8 × 12 ins. for the struts. The bents were spaced 8 ft. apart, except the two end bents, which were canted so as to loosely fit the angular plan of the wing walls, and were made less than 8 ft. from the next bent. The slope of the upper face of the dam was made 1½ to 1 in order to secure more downward pressure than horizontal thrust, and thus prevent sliding. An additional precaution was also taken to prevent sliding by setting three pins of 1½-in. iron behind three of the sills about the center of the lock. The holes for these were drilled after the frame had been weighted, and rested on the bottom, and were bored through a 2-in. pipe with hand drills from a barge moved to the upper side of bulkhead. The result showed these precautions might have been omitted, as the framework never slid at all.

The rafters or caps supported purlines of 6 × 12-in. timber, shown in the accompanying plates, and on these, when the framework was settled in its place on the bottom, the apron of 3 × 12-in. plank was spiked.

The original plan was to use purlines 16 ft. long and butt them on the caps, as shown in Plate LXIV, Fig. 4, but as 18-ft. timber was of much greater use in our barge repairs, it was concluded to use 18-ft. timbers and let them overlap, as Plate LXXV clearly shows. A single ⅝-in. screw bolt secured the purlines to each rafter, the hole being bored a little large, to facilitate the dismemberment of the structure when it had served its purpose. The struts were fastened to the rafter and sill with 2-in. plank nailed to each.

No "gaining in" was used, the idea being to cut the timber as little as possible, in order to keep it for other purposes. At the lower end of the rafter a 1-in. bolt was passed through it and the sill, and a key of oak wood driven in, to prevent the rafters sliding on the sill, owing to the resultant of the normal water pressure and the angular position of the struts. The structure was lightly braced diagonally, both in plan and on its vertical members, to prevent deformation while being placed in position.

The plan, Plate LXXIV, shows these of 2 x 12-in. plank. Plate

LXXV shows much lighter bracing, being about 2 x 6-in. scraps picked up in the boat yard. It was designed that the structure should have considerable flexibility, and the purlines were kept short for this purpose. The vertical diagonal braces were made light, so that they should give way if necessary and allow the bents to adapt themselves to possible inequalities of the bottom.

Careful soundings showed little variation in the bottom however, and when the water had been removed the bents were found to rest on a smooth concrete floor, kept perfectly free of all deposit by the powerful scour from the discharge culverts.

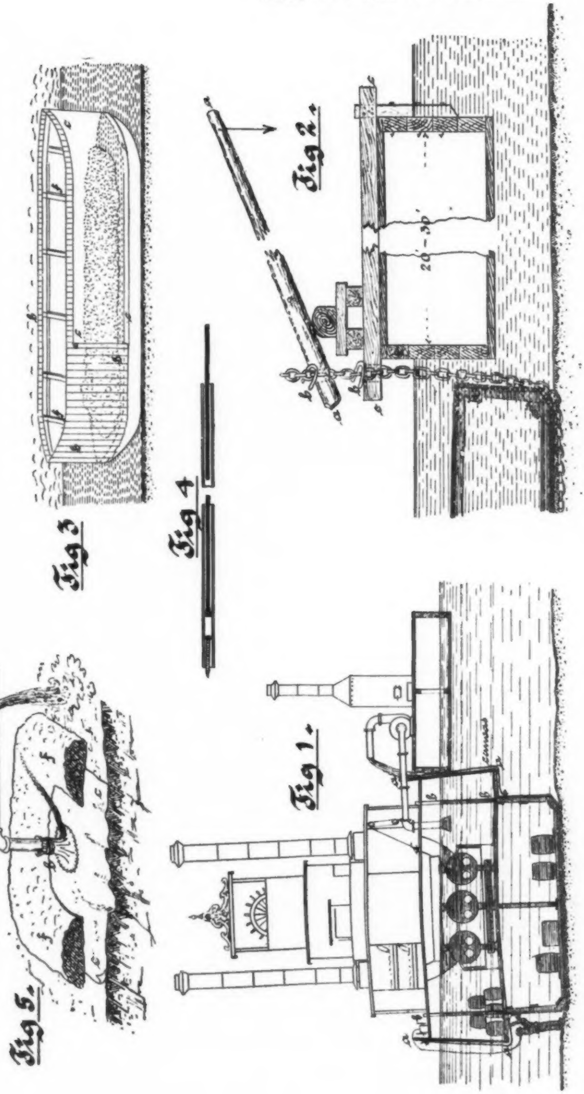
The thirteen bents and the purlines were set up in the United States Dry Dock, 2½ miles up the canal, and floated down to place by means of a tow boat and two flat boats, suspended between the latter by six ⅝-in. chains. The frame, in getting out of the dock, had to pass over 4½ ft. depth of water, owing to the low stage of the river, and the strain on the high side of the structure carried by three chains was considerable.

Fig. 2, Plate LXXVI, shows the device used to suspend it.

No difficulty, however, marked its journey from the dock to the site of its final location; it was a clumsy thing to tow, and it was urged along gently, to prevent any distortion *en route*. Arrived at the proper place, it was floated over the marks made beforehand, and readily sunk with old rails. The chains enabled it to be controlled as they were slacked off from the barges, and, in case of an error, to be raised and moved slightly, should it be necessary.

About 60 000 pounds of old rails were borrowed from a friendly railroad, and used in sinking and ballasting the structure. The spaces between the purlines were admirably adapted to receiving and holding the rails, and the rails were skidded into place from the lower barge, one at a time, the sinking being kept uniform throughout. A portion only of the rails were required to sink the frame; the rest were added to overcome the buoyancy of the 3-in. covering presently to be put on. The load put on was solely to overcome buoyancy. None was used to give the structure further weight, the water being calculated to hold it down sufficiently. Up to this point the work had been carried on without the services of the diver, who arrived in time to make an examination of the bottom and see that no rough points or boulders interfered with the proper contact of sills and floor.

PLATE LXXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 707.
MEIGS ON CANVAS FOR BULKHEADS.



During this stage of the work the depth of the water was between 9 and 10 ft. All the ballast of rails having been put on, the first tier of sheathing plank, 3 x 12 ins. and 16 ft. long, was added rapidly. The diver, stationed on the bottom, slid each plank close against its neighbor and saw that it touched bottom properly.

Two spikes were used to secure it to the lower purline, and one spike was used at the upper end. The spikes below water were started before sending the plank down, and driven with a "shot gun" from the barge above (Plate LXXVI, Fig. 4). The shot gun consisted of a $\frac{3}{4}$-in. pipe with an iron rod in it for a hammer. All the diver had to do was to place the shot gun on the spike and give the signal by rapping on the pipe, when two or three strokes of the rammer sent it home. The upper end of the plank was out of water generally, and the spike in that end was driven by an ordinary hammer.

We found it very important that this upper end should be out of water, and wished this first tier had been 18 instead of 16 ft. long for that reason. In a few places the spike had to be driven when the upper end was 3 or 4 ins. beneath the surface, and disagreeable splashing resulted. The second tier of plank was made 8 ft. long and placed as the lower tier progressed. Had the water required it, a tier of short 1-in. plank would have been used on the upper purline, and a narrow strip of canvas laid over it; but this was held in reserve and was not, as it happened, required.

The canvas sheet, which was designed to give tightness to the apron, was of two breadths of 10 ft., and one breadth of 6 ft., wide, sewed together edge to edge for convenience, and about 4 ft. longer than the extreme length of the apron. Some old $\frac{1}{2}$-in. and $\frac{5}{8}$-in. chain was sewed to one edge continuously to act as a sinker and insure the lower edge of the canvas sheet hugging the bottom tightly. A few stones laid on it would have answered the same purpose, but not so well. The canvas was 12-oz. duck.

The sheet was spread under water by the diver. It lapped on the bottom about 12 ins., covered the face of the apron and extended some inches up the face of the wing walls at the end of the dam. Cleats were nailed on the angle between the apron and the wing walls; these were of 1 x 4-in. strips, nailed with 2-in. wire nails about 12 ins. apart. The upper edge of the canvas was also lightly cleated to the planking in a similar manner. No other nails were driven in the canvas, which was designed to be cut up into tarpaulins eventually.

Where the plank touched bottom no beveling of the end was done. It was intended originally to use "stop waters," as at *A*, Fig. 3, Plate LXXIV, of 1 x 12 ins., 18 ins. long, to make a close joint. In one place, where there was a ragged hole in the rock bottom, these beveled stop waters were used, and, had the bottom been less regular, would have been applied all along the junction of the apron with the bottom. The diver thought them unnecessary, except in the one place mentioned, and the result showed he was right.

All being in readiness, the pump was started and the immediate lowering of the water behind the dam showed that the canvas had taken hold at once. About 3 ft. of water were taken out before night, and the pumping operations suspended till next day, in order to avoid accidents. About six hours' pumping in all laid the lock chambers dry. The placing of the dam in position began November 20th, and on the 25th it was pumped dry. The diver was employed five days in all. Some of these days it was so cold he could not work continuously and had to come up, take the suit off and warm himself. The weather was cold and freezing. The river closed two days after the lock was pumped out.

The tightness of the dam astonished everybody. Large quantities of fish, the writer regrets to say, found themselves deprived of their natural element, and were carted off by the barrelful. The small fry were a serious nuisance by clogging up the pulsometer.

The length of the dam was 96 ft. on the lower edge of the apron, and about 8 ft. less at the back. No drip was seen on the under side of the apron, nor could any water be discovered coming under the bulkhead where it joined the bottom. All leakage could have easily been pumped by hand with one hand pump, and even that came through open joints in the masonry around the ends of the bulkhead and not under the canvas. A pulsometer, with 3-in. discharge pipe, worked intermittently, kept the lock chamber dry. Some of these leaks, which at first seemed serious, were cured by dumping rotten stable manure in their vicinity, a load of manure having been provided beforehand for this purpose.

One could walk under the structure anywhere without getting wet.

At the beginning of operations and while the sheeting was being put on, the depth of water varied between 9 and 10 ft.

The dam remained in position from November 25th to December

8th, when, repairs being completed, the water was admitted and the dam removed. The sheeting was easily taken off after the removal of the canvas, which latter was cut up and converted into tarpaulins. The rails were hooked up, one end at a time, and hoisted out by means of a steam winch and a gin pole on the lock wall.

Not a rail was lost. The wrecking of the rest of the structure was easy, and all of the timber was saved for other purposes. The spikes used were ¼-in. wire nails, 6 ins. long, and were easily pulled out.

The whole work was in marked contrast to our mud dam experiences, where the dam was turned over after use to the tender mercies of a dredge, that made kindling wood of all timber. The entrance to the lock was also left perfectly clean.

Some anxiety was felt as to the effect of the ice on the canvas. It was thought it might freeze to the canvas, and, as the river raised or fell, tear it off, but no such an effect was seen. It froze tightly to the plank beneath and took care of itself very well.

The greatest head of water that the bulkhead withstood was about 12 ft. This was after the river had closed. There was no indication that the canvas would not have withstood 25 ft. of head just as well. The pressure of the water forced the canvas into all depressions and cracks and increased the tightness with the pressure.

An apron of poles would probably have answered every purpose to support the canvas. The tendency of the water to force it into all openings overcomes all but the grossest defects of workmanship.

The structure described above was on a smooth rock bottom, but it would probably work equally well on gravel, care being taken to give a sufficient surface of canvas, say 4 or 5 ft. on the bottom, to get the required resistance to the infiltration of water.

The bulkhead at the lower lock, with its 12-ft. head of water on it, showed that, with that amount of head, the seepage was small. How much greater head 12-oz. duck can withstand without allowing much water to pass through is not generally known, the writer thinks. The following experiment was tried to determine this point.

A piece of 4-in. steam pipe was fitted at one end with a pair of common flange coupling plates, between which a gasket of 12-oz. duck was clamped, closing that end of the pipe. The other end was reduced, and connected with a small force pump, and a pressure

gauge. The figures below show what happened when the pressure was applied :

Pressure, 1 lb., canvas shows no signs of any water coming through.

" 2 lbs., water comes through the pores in rapid drops, so that rapid pumping is required to keep up pressure.

" 5 lbs., water comes through so fast that the pump maintains this pressure with the greatest difficulty.

Supposing that mud might have something to do with making it tight, for no leakage at all was observable in the dam, already described, due to seepage through the canvas, the 4-in. pipe was opened and a couple of handfuls of mud placed therein, and shaken up with the water. Pressure was again applied with the following result :

Pressure, $2\frac{1}{2}$ lbs., canvas perfectly water tight.

" 5 lbs., slight drip in a few places, but water comes through clear.

" $7\frac{1}{2}$ lbs., same as above, a little more leakage.

" 10 lbs., still more seepage, but not large.

" 50 lbs., water still comes through clear.

Leakage through canvas about as in first experiment with 10-lb. pressure.

The canvas was still unruptured, though supporting a pressure on a circle of $4\frac{1}{2}$ ins. diameter, in the neighborhood of 800 lbs., without tearing.

This experiment goes to show that muddy water is favorable to the tightness by closing up the pores of the cloth.

In very clear water either heavier canvas or treatment of the lighter grades with some filler, like linseed oil, would be necessary to prevent the leakage from assuming troublesome proportions in a dam or bulkhead depending on canvas under heavy pressure.

In removing the canvas from the lower lock bulkhead, the side toward the water was found to be covered with $\frac{1}{4}$ to $\frac{1}{2}$ in. of mud, though the water was generally pretty clear while it was submerged.

It had evidently filtered this out of the water. The side toward the plank was perfectly clean, the mud being all collected on the other side.

This action of canvas is made use of in well-known filters.

The writer has assisted at the raising of several sunken steamers, when canvas played the most important part. The diver's first care is to close the break in the hull by nailing planks across, and over this a patch of canvas of proper size, fastened to the hull outside the plank with laths, using $1\frac{1}{2}$-in. wire nails.

Then a bulkhead of vertical plank is built around the boat like a fence, the upper ends projecting above water, and the lower ends spiked to the edge of the hull. No particular care need be taken to make this planking tight; rough boards are used just as they come. The sheet of canvas, wide enough to lap over the lower ends of the boards and be battened to the hull 4 or 5 ins. from the ends of the boards and to reach above water, is usually in one width, made by sewing the necessary breadths together. It could be in several widths lapping on each other and secured with laths nailed over the joints; but this greatly increases the work of the diver.

The writer has seen a coal barge (Plate LXXVI, Fig. 3) 200 ft. long, 25 ft. wide, loaded with coal, raised in this manner.

The barge began to float so soon as 4 ft. of water was taken from the inside of the bulkhead, so that the head of water on the bulkhead was never very great.

On another occasion the writer witnessed the raising of a large Mississippi River steamer 240 ft. long and 26 ft. wide (Plate LXXVI, Fig. 1). This boat was pretty heavily loaded and had a hole 2 x 4 ft. made in her bow by a snag, and sank in a few minutes. The bulkhead planks in this case were nailed on her guards, 3 ft. beyond the hull. The canvas was secured to the hull 12 to 18 ins. below the line of her gunwale. The under surface of her guards was naturally very uneven, consisting of 3 x 6-in. outriggers or floor beams, on top of which the deck plank rested, but the canvas found its way into all these uneven places and made all tight. This boat began to raise when but 18 ins. of water was removed from the inside of the bulkhead so that the pressure was light. Had it increased to 5 or 6 ft., the water would probably have torn her guards off by lifting them up.

It sometimes happens that a boat settles down in soft mud. When this is the case, a much greater lifting power is required to tear it from the bottom, which attaches itself to the boat like an enormous sucker.

In one case the boat had sunk so there was 8 ft. of water on the

main deck, and this deck was laid bare before it would leave the bottom. It is needless to remark that when the bottom did let go, the boat bounced up to the surface with a quickness that threw every one on board off their feet.

Where the boat settles down on sand no such difficulty occurs. It would, of course, on a mud bottom facilitate operations to direct a stream from a hose under the bottom so as to break the connection with the boat, and allow the water to get in.

Plate LXXVI, Fig. 2, shows the usual method employed in raising a sunken structure (in this case a boat) by means of barges and chains. It is not new, but is simple, operates perfectly, and is useful to know. The sketch shows pretty well how it is worked.

a a is a stout lever of wood. In steamboat raising on the Mississippi by chains, usually an oak or elm tree is used, of 12 ins. at the butt, and as long as the width of the barge will permit. *b b* are two U-shaped toggles, of square iron, of such a width as to slip easily on the chain, but too narrow to allow the chain to pull through. Depressing the lever draws the chain up through the hole in timber *c c*, and the toggle is then slipped on the chain one or two links lower down.

Raising the lever, the chain above the lower toggle is slacked, and the upper toggle is moved one link lower. Tremendous strains can be applied to the chains in this way, sufficient to break the chain or lift the load. This method of raising boats is employed when no diver is available and the boat is small. Also in deep water it is used when a bulkhead with its top out of water is impracticable.

It is hard on the boat and most laborious, and is seldom employed on the Mississippi on a boat of more than 150 tons.

It only requires some chains and toggles, which can be made anywhere, the rest of the tools being taken from the woods near by.

Plate LXXVI, Fig. 3, shows the coal barge in process of raising. In Fig, 3, *b b* is the bulkhead plank represented at one end without the canvas *c c*, which is shown, being unrolled along the side. *f f* is a frame to nail the upper ends of bulkhead to. Fig. 1 represents the steamboat raising, where the bulkhead was secured to the guards at *a*. Fig. 1 (*b*) shows the position of the bulkhead when the guards are considered too weak to stand the strain of the other method. It is much more troublesome. Usually freight has to be got out of the way, and a cleat nailed on the deck to foot the plank against.

It should be remembered that the canvas skin in all these operations is what gives the element of water tightness. All the rest is only a frame on which to stretch it.

Of course a large hole might allow the canvas to bag in it so as to be pulled bodily through and rupture the canvas, or so disarrange it as to cause a leak somewhere else.

The ideal is a basket with a skin of canvas outside. The water finds every leak, and if the canvas covers the leak, even, if a very small one, it pushes the cloth in and stops the hole.

Plate LXXVI, Fig. 4, is a sketch of the "shot gun" mentioned before. It is invaluable in such cases as the bulkhead at the lower lock. There is nothing new about it, but it is given, as it may have novelty for some people.

Plate LXXVI, Fig. 5, is a suggestion for stopping springs in foundations that are to be water tight. *c c* is a sheet of canvas of proper size, in the center of which a patch of cloth is sewed on, a sort of boot (*b*) making connection by wrapping it with wire or cord with a suitable discharge pipe. Beginning at the edges of the canvas, concrete (*f*) is dumped on and rammed down firmly, crowding all the water toward the center, and allowing it to escape through the discharge pipe *d*. The lower this discharge pipe is kept, of course, the less head will be felt under the canvas. The cloth completely protects the concrete from washing, and, when set, and all filled up around the pipe, the stopper *e* is driven into the pipe, and the leak is overcome. This method seems to have some advantages over cement or concrete in sacks, and the cement is equally well protected. The close fitting of the canvas to the bottom is not a matter of surprise, or that it should be sufficiently tight to exclude water from passing beneath it; for instance, on a gravel bottom. If the head be 12 ft. for example, each square foot of canvas pressing on the bottom will have a pressure on it of $12 \times 62.3 = 748$ lbs., quite enough to insure a tight fit.

It is observable that when the pressure comes upon these canvas-covered bulkheads, the cloth fits so closely as to show the slightest inequality of the surface beneath, the total pressure, of course, if the dimensions of the bulkhead be large, being measured by many tons. On the lower lock bulkhead, the total pressure on the canvas measured over 300 tons.

The following is the cost of this bulkhead, including all cost of putting in and removing:

Lumber	$374 73
Canvas	95 00
Pipe	8 16
Diver	173 12
Nails	21 00
Labor	486 99
Total	$1 159 00
Deduct material saved	400 00
	$759 00

The bitter cold weather, of course, increased the expense. The river was frozen up when the bulkhead was removed, and the rails had to be hoisted up the lock walls and hauled to the railroad by a circuitous route. The cost of loading and unloading the rails at the railroad yard, whence they were borrowed, was considerable.

The driver was employed only five days, the removal being accomplished without his help.

DISCUSSION.

S. BENT RUSSELL, M. Am. Soc. C. E.—Mr. Meigs' interesting paper recalls a case in my experience during the fall of 1890.

In building the new inlet tower of the low-service extension of the St. Louis Water Works, it was necessary to sink a coffer-dam on rough rock bottom. The dam was rectangular and about 26 x 62 ft. inside and 20 ft. high. The depth of water ranged from 11 to 20 ft. during construction.* The lower courses of the coffer-dam were framed together on the river bank and then floated to position.

As near as I can remember, our experience with canvas was as follows:

As it had been decided to try canvas to close the joint around the base of the dam, the contractor attached a canvas curtain to the out-

* For further information about this work see *Engineering News*, July 4th, 1891.

side of the structure. The upper edge of the curtain was secured to the wall several feet above the lower edge of the coffer-dam.

The intention was to lay the lower edge of the curtain on the rock bottom and weight it down with stones. While the dam was being floated into position, the curtain was rolled up and held by light fastenings.

When the coffer-dam had been sunk to its place, divers were sent down to spread the curtain. They reported that the canvas had become so badly damaged as to be worthless.

At the time this destruction was thought to be due to the fluttering action of the swift current in the channel of the Mississippi, and, perhaps, also, to the cutting action of sand carried by the stream.

It is possible that this result might have been avoided by proper precautions.

The coffer-dam was finally made tight by the use of bags of concrete and of clay, but the expense and delay were excessive.

In view of this experience it would be well to take extra precautions in using canvas in swift currents. As Mr. Meigs makes nc mention of trouble from swift currents in the use of canvas, I have brought out this case in the hope that he will give further information bearing on the point.

Montgomery Meigs, M. Am. Soc. C. E.—The experience of Mr. Russell with a canvas curtain for making a tight joint between coffer-dam and rock was unfortunate, but the cause of the failure is not explained. Possibly the canvas was torn against a barge or some such obstacle. I do not think, if properly secured, the water would tear it.

The chain we sewed to the edge of our curtain was much better for obvious reasons than weighting with stones. On one occasion the water fell outside the dam described, and the pressure inside (the dam being at the time full of water) lifted the structure like a valve, letting the water pass underneath. The canvas and sustaining framework resumed their position perfectly as soon as the pressure was equalized, and left the tightness unimpaired.

When we put the coffer-dam in, there was practically no current and no trouble in spreading the sheet of canvas. In swift water, extra precautions would have to be taken, the canvas rolled up carefully until the time of spreading it came, battens nailed on wherever it showed a tendency to bag in the current. The divers spread many such sheets of canvas on wrecks in the Mississippi every year. Some of them 12 ft. wide and over 200 ft. long, and in a swift current.

At one wreck I assisted in raising, the bottom of the boat was ripped for 40 ft., leaving a hole 2 ft. wide in places. The diver, after the boat was raised by the usual method of surrounding the hole with a canvas and plank coffer-dam and floating the boat, went underneath her bottom, nailed 1-in. boards across the break, and spread over that a

sheet of canvas well battened. This allowed the boat to be pumped dry, and she made a trip of over 100 miles up stream, to be docked and repaired.

A swift current will, of course, always greatly increase the difficulty in placing a coffer-dam of any kind. I think that with a due appreciation of the difficulty beforehand, canvas can be safely and economically used, as Mr. Russell designed doing at the St. Louis water works crib.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

708.

(Vol. XXXI.—May, 1894.)

GENERAL NOTES ON THE GREAT KANAWHA IMPROVEMENT, TRIPPING BARS AND IMPROVED HURTERS ON CHANOINE WICKET DAMS, ETC.

By ADDISON M. SCOTT, M. Am. Soc. C. E.

READ MAY 2D, 1894.

WITH DISCUSSION.

The following was in the main prepared as a discussion on the paper of F. Guillain, Inspecteur Général des Ponts et Chaussées, on "Navigation Works Executed in France from 1876 to 1891," *Transactions*, Volume XXIX, No. 604, July, 1893, but being too late for the discussion, is now, at the suggestion of the Secretary, presented as a separate paper.

Reference is made on page 16 of the paper referred to, to the use of the Pasqueau hurter by Colonel Merrill, on the Ohio, at Davis Island Dam. As the first use in America of this valuable improvement in Chanoine dams was on the Great Kanawha, and the most extended application of its leading principle either in Europe or America has been on the same river, an account of the experience with it on the Great Kanawha seems worthy of record in this connection.

Only a brief outline of the Great Kanawha improvement will be attempted in this paper. The United States Engineer Department has recently published full detailed drawings and descriptions of lock and dam No. 7, one of the last movable dams completed, and also of lock and dam No. 2, one of the two fixed dams in the upper part of the river. The former is accompanied by a general history and description of the river, and of its commerce and improvements, with cost of construction, operation, etc. This is also published as part of the report for 1892, of Colonel William P. Craighill, the officer in charge of the work.*

The improvement of the Great Kanawha River will consist of ten locks and dams, carrying the slack-water from the mouth of the river, a distance of 90½ miles. The two upper dams of the system (Nos. 2 and 3 of profile) are fixed, and the remaining eight, numbered from 4 to 11, inclusive, are "movable," being kept up in low and down in medium and high stages of the river.

The improvement is designed mainly for a coal commerce, the barges for which are mostly 25 x 130 ft., with draft of 6½ ft., and capacity of 500 tons. The minimum low-water depth on mitre-sills is 7 ft.

Some of the important features and dimensions of each lock and dam are given in the following table:

Number of lock and dam.	Style of dam.	Maximum lift in feet.	Length of dam. Feet.			Lock dimensions. Feet.		Location, miles from mouth.	
			Navigation pass.	Weir.	Total.	Clear width.	Length between quoins.		
No. 2..	Fixed.	12	....	......	524	50	308	85	Finished in 1887
No. 3..	"	12	....	......	564	"	311	80	" 1882
No. 4..	Movable.	7	248	210	458	"	300	73¾	" 1880
No. 5..	"	7	250	265	515	"	300	67¾	" 1880
No. 6..	"	8¼	248	310	558	55	342	54¼	" 1886
No. 7..	"	8	248	316	564	"	"	44¼	" 1893
No. 8..	"	8¼	248	292	540	"	"	36	" 1893
No. 9..	"	6½	248	300	548	"	"	25½	Begun in 1893
No. 10..	"	7½	248	292	540	"	"	18¾	" 1893
No. 11..	"	10	300	372	672	"	"	1¾	" 1893

Locks and dams Nos. 4 and 5 were completed and put in operation in July, 1880, and were the first movable dams for slack-water improve-

* Report Chief of Engineers, 1892, III, 2053.

ment in America.* Five others, three movable and two fixed, as shown by the table, have been built since 1880, and three more movable dams, carrying the improvement to the mouth of the river, are now under construction.

These eight movable dams are all of the Chanoine wicket type, both on navigation passes and weirs, operated from service bridges. The pass wickets are 3 ft. 8 ins. wide—4 ft. between centers—and from 13 ft. 5½ ins. to 14 ft. 1 in. long. The vertical heights (with two exceptions, one of which is 12 ft. 6 ins., and the other 12 ft. 9 ins.) are all 13 ft. above pass sills. These wickets, with the exception of the La Mulatière dam at Lyons, France, where the vertical height above sill is 13 ft. 1¼ ins., and length of wicket 14 ft. 3½ ins., are the largest ever built.

The navigation passes at dams 4 and 5 were built to be lowered by tripping bars operated by gearing wells in the lock wall and center pier on the regular Chanoine plan so extensively practiced on the narrower passes in France. The bar operated from the lock tripped 34 wickets, and was 140 ft. 10 ins. long; the one from the center pier was 119 ft. 2 ins. and tripped 28 wickets. A good deal of trouble was had with the tripping bars, from the first. Besides the difficulties unavoidably connected with them, it was found that both bars and gearing had been made too light for such wide passes. As was to be expected. the bars became clogged more or less with water-soaked sticks, cinders, gravel, etc.; when much clogged the gearing was not strong enough to pull the bar in (the wickets were, of course, tripped by the pull), and they were sometimes buckled and broken in being pushed back to place. Proper provision had not been made for keeping the bars on the guides, and they were frequently found off the track when the dam was to be raised. Any one particularly interested in the subject will find quite full reports of the manœuvres of the Great Kanawha dams each year since their completion, with accounts of the difficulties met with, in Col. Craighill's annual reports to the Chief of Engineers.

On account of the trouble with the tripping bars at dams 4 and 5, the writer, as assistant engineer on the work, in a report made in June, 1881, recommended a trial of the Pasqueau hurters on one of the bar sections, with a view, if successful, of adopting them and eliminating the tripping bars altogether.† This was approved, and the same fall

---

* The Davis Island dam was completed in 1885.

† Report Chief of Engineers, 1881, I, 914.

the bar on the lock side at No. 5 was taken out and some of the adapted Pasqueau hurters (Fig. 3, Plate LXXVII) put in. They were entirely successful, and since then the pier bar at No. 5, and both lock and pier bars at No. 4, have been shortened and improved hurters put in. In making the change, the Chanoine hurters were removed and the new hurters placed by divers (the regular lock hands doing the diving) without a coffer-dam. No change was made in the "slides," the same ones being used as with the Chanoine hurters.

At dam No. 5 on the pier section of the pass, 16 wickets are now tripped by the bar; at No. 4, 13 wickets on the pier and 14 on the lock section are tripped by the shortened bars. These short bars give comparatively little trouble, and, being of considerable advantage in lowering, have been retained on these first dams. It may be stated here that while the decided superiority of the Pasqueau principle has been thoroughly established, experience on the Kanawha proves that it would be practicable to operate a 250-ft. pass, and somewhat wider—up to at least 300 ft., it is believed—with properly constructed tripping bars. The difference in first cost between tripping bars and the improved hurters is comparatively small, being about $2 000 in favor of the hurters (allowing for the bars and gearing to be made strong enough) on a 248-ft. pass. While the dam can be lowered faster and easier with the bars, the difference in time is not enough ordinarily to be material. The 62 wickets on one of the Great Kanawha passes, fitted entirely with the automatic hurters, are usually lowered in less than an hour. Generally about 45 minutes are taken for this part of the manœuvre; with the bars working well, they could be lowered in 10 or 15 minutes.

The great advantage of the Pasqueau principle is in time and labor saved in raising the dam. With the tripping bars, under the most favorable conditions, it is generally necessary to clear the gearing wells of deposit, and always advisable to examine the bar and hurters throughout to see that they are clear and the props down in the hurter seats in position to trip. The greater part of this work must be done by divers. At dams 4 and 5 on the first raising in the spring, when there is always considerable deposit on the foundations and in the wells, it usually took three or four days, and sometimes longer, to clear and adjust the bars and gearing to get ready to raise. After the first raising in the spring, from one to two days were generally required for this. With the improved hurters this is all avoided, and the dam in conse-

PLATE LXXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 708.
SCOTT ON MOVABLE DAMS.

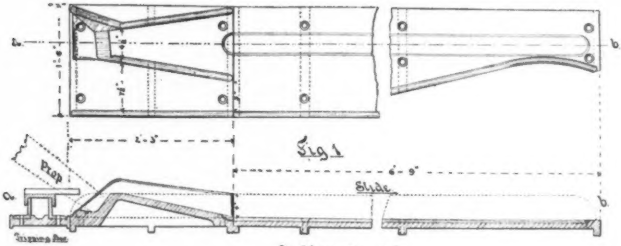


Fig 1

Section on a.b

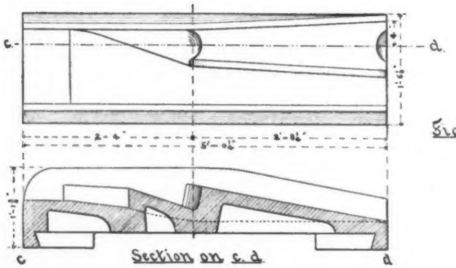


Fig 2.

Section on c.d.

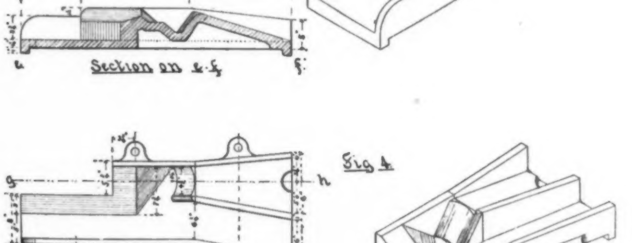


Fig 3.

Section on e.f.

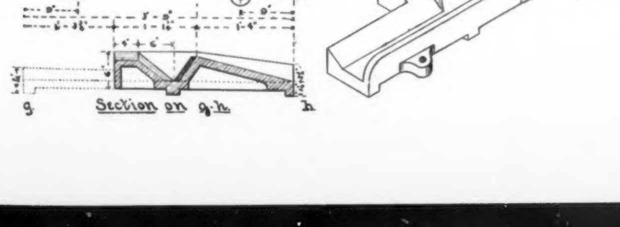


Fig 4.

Section on g.h.

quence put up quicker and easier and on a higher stage of water than with the bars.

On the whole the advantages of the automatic hurters are so decided that it seems safe to say that the day of tripping bars on new Chanoine dams is past, save, perhaps, in short lengths (as at present at dams 4 and 5), and in exceptional cases, where rapid lowering is of great importance.

Without departing from the leading and important feature of Pasqueau's stepped hurter, namely, that of disengaging the prop from the hurter-seat by a short up-stream movement of the wicket, the form of the hurter has been considerably modified on the Great Kanawha. Different forms of hurters are shown on Plate LXXVII.

Fig. 1, Plate LXXVII, shows a regular Chanoine hurter in plane and section. This is the hurter used on dams 4 and 5 with the tripping bars, and is substantially the same used on the Chanoine dams in France. The tripping bar and the end of a prop are represented in the section.

The "slide" or guide below the hurter, shown on Fig. 1, is practically the same with all the hurters. The hurter and slide are often cast in one piece.

Fig. 2, Plate LXXVII, is the regular Pasqueau stepped hurter copied from an official drawing of part of the La Mulatière dam, dated October, 1879, about two years before the completion of that work.*

Fig. 3, Plate LXXVII, shows the modified Pasqueau hurter first used on the Great Kanawha on parts of the navigation passes at dams 4 and 5. The first were placed, as before mentioned, at No. 5, in 1881. The block or stop *x* on the top of the step was required at these dams to insure a short defined run of the prop. This was necessary on account of the nearness of the horse, when upright, to the top of the sill—the horse in this position having an inclination up stream, as usual in tripping-bar construction. The stop proved an advantage in reducing the time to lower and this form was adopted for dam No. 6, though not necessary on that work, owing to the down-stream inclination of the horse (see Fig. 5, Plate LXXVIII).

Fig. 4, Plate LXXVII, represents the "shunting" hurter adopted throughout on dams 7 and 8, both completed last year, and proposed

* The Pasqueau hurter was patented in the U. S. in 1880.

for the three remaining dams now under construction. Some of these hurters are also in use on dams 4 and 5; the first were placed for trial at No. 4, in 1888. Other forms of the shunting hurter are also in use at No. 4, but it does not seem necessary to represent them here.

The operation of this hurter is apparent from the drawing. When the wicket is pulled up stream the end of the prop is carried or shunted directly away from the seat, on the level floor of the hurter, into the descending channel, in position to let the wicket go down. This hurter works as well in every respect as the stepped form and has some, though not highly material, advantages over it. It being lower than the stepped hurter (it is necessarily no higher than the Chanoine hurter) it lets the wicket lie closer to the platform when down. It is also simpler in form and somewhat lighter than the stepped hurter.

The manner of lowering the pass wickets followed on the Great Kanawha was devised the first season the Pasqueau hurters were used on this river, and will be briefly described.

In lowering the dam as first practiced with the improved hurters, the wickets were put on the swing (*en bascule*) and then pulled up stream far enough to free the end of the prop from the hurter seat and then lowered—all being done by the winch and butt chain. The Kanawha practice is to pull the standing wicket by the top with a clutch and line till the prop is free, then to disengage the clutch and let the wicket fall. Both ways of lowering will be understood from Plate LXXVIII, representing a general cross-section of the navigation pass of dam 7. The clutch is simply placed over the top of the wicket. It has two lines as shown, the main line going to the drum of the manœuvring winch, and a smaller one attached to the upright piece of the clutch and held in hand. As soon as the wicket is wound far enough up stream to free the prop, the small line is snubbed to the service bridge and the main line released of its strain by unwinding a little on the winch. This instantly disengages the clutch and the wicket falls. This is a much better way to lower than by the chain. It can be done in about one-fourth the time, saving from two to three hours in lowering the pass (enough frequently in a quick rise to be of great importance), and avoids the risk of water-soaked sticks and drift being drawn against the horse when the wicket is swung, and preventing the latter from going down flat to place behind the sill. No injury is ever done the wickets or any of the parts by letting them fall in this way, even

PLATE LXXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 708.
SCOTT ON MOVABLE DAMS.

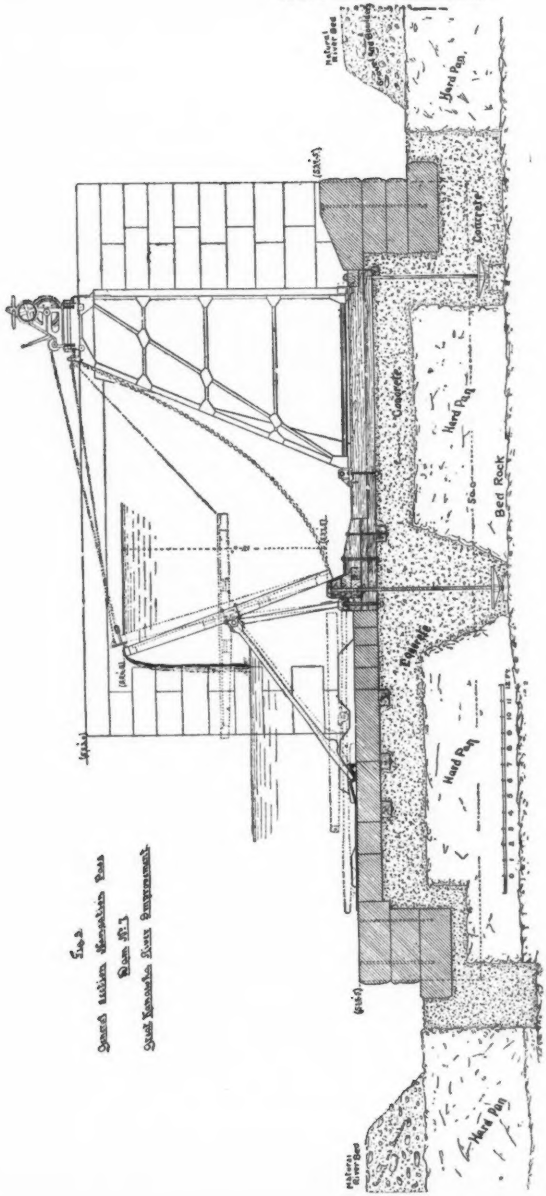


Fig. 2
Cross section Navigation Pass
Dam No. 3
Great Kanawha River Improvement

in low stages with full head. The wicket starts rapidly, of course, when first released, but the water below soon checks it, and it settles down on the cushions without any noticeable shock.

The following from the published description, before referred to, of lock and dam No. 7, and of the river and general improvement, will perhaps be useful here. The description was written in 1892, before the completion of locks and dams Nos. 7 and 8.

"The experience with movable dams on this river has on the whole been very satisfactory. They are easily and rapidly manœuvred (in these respects dam No. 6 and those now under construction have considerable advantage over those first built), the expense of operation and maintenance is but little, if any, more than with fixed dams, and they prove highly satisfactory to the river interests.

"The movable dams are kept up whenever there is not water enough in the river for coal-boat navigation and down at other times. Their advantages over the ordinary fixed dams for a commerce and river like the Great Kanawha are decided, furnishing the benefits of the usual slack water without its most serious drawbacks. With fixed dams everything must pass through the locks; with them navigation is entirely suspended, too, when the river is near or above the top of the lock walls. With movable dams the locks are only used when the discharge of the river is so small as to make them necessary. At all other times they are down, practically on the river bottom, out of the way, affording unobstructed open navigation. This is of great advantage to all classes of commerce, and is particularly so with coal, transported, as it is, and empty barges returned, in 'fleets' of large barges. More barges can, of course, be taken by a towboat, and much better time made by all kinds of craft in 'open river,' when there is water enough for such navigation, than when the stage or discharge compels the use of the locks.

"The movable dams being down in high water, there is comparatively little difficulty in protecting the banks about the works from scour. In this respect they have considerable advantage, too, over the fixed dams.

"Experience with the dams has naturally suggested improvements, and No. 6, the last one completed, has considerable advantages over those first built in strength and durability of construction, facilities for rapid manœuvring, and cost of operation and maintenance. Dams 7 and 8 have been still further improved in some of their details.

"No. 6 has been in operation over five years. The average cost of operating and maintaining the lock and dam has been $2 515 per year. This covers wages, supplies, repairs, including considerable addition to the rip-rapping, and all expenses connected with the work. The entire cost during the five years of repairs on the dam proper and on

all of its apparatus, including paints, one of the principal items, has been something less than $250, or an average of $50 per year.

"This dam is put up by four or five men in from 7 to 12 hours; the usual time is about eight hours. It is lowered with the same force in about two hours. No material difficulty has ever been met with in any of the manœuvres at No. 6.

"Four men are employed regularly at each work, the same as at the fixed dams. In raising and lowering the dams, one or two extra men are often hired."

The writer has been collecting notes for a paper on the Great Kanawha works of more technical interest, especially relating to experience in construction of the locks and dams, which he hopes to have the honor of submitting to the Society at a future day.

DISCUSSION.

WILLIAM P. CRAIGHILL, Pres. Am. Soc. C. E. (by letter).—The last paragraph,* page 16, of Mr. Guillain's paper on "Navigation Works in France" is calculated to give an erroneous impression and for this reason I have concluded to add a few words to Mr. Scott's paper. I have hesitated to say anything because the Pasqueau hurter or other device for manœuvring movable dams is quite limited in application, and the subject may not be of very general interest to the engineers of this country at present.

My first knowledge of the Pasqueau hurter was gained in 1878, when I saw a model of it in M. Pasqueau's office at Lyons. I was visiting that city with Colonel Merrill and Mr. W. R. Hutton, to obtain information to be applied in the development of the use of movable dams on the Ohio and Great Kanawha Rivers. I was very favorably impressed with the Pasqueau hurter, but did not think it of sufficient importance to justify a revision of the plans of dams 4 and 5 on the Great Kanawha, which were then, and had been for some time, in process of construction. These dams were provided with the usual tripping bar of the Chanoine system, which had answered well for navigation passes of moderate width.

The tripping bar on the Kanawha did not turn out as well as was expected, owing mainly to want of due proportion of parts and consequent want of stiffness and strength. It was therefore decided, some time after my return from Europe, to make a trial of a few of the Pasqueau hurters on the Great Kanawha. This was done, and, as Mr.

---

* *Transactions* Am. Soc. C. E., Vol. XXIX, p. 1.

Scott remarks, "the first use of this valuable improvement was on the Great Kanawha, and the most extended application of its leading principle, either in Europe or America, has been on the same river."

M. Pasqueau had his invention patented in this country, and, through agents, made a claim for a large royalty. His claim was considered excessive, especially as his invention had been used without charge in France and its experimental introduction in this country had largely advertised it. A reasonable offer was made for the use of the invention in this country, and declined. Whereupon suit was brought in the Court of Claims by M. Pasqueau's agents.

These difficulties as to the continued use of the Pasqueau hurter and further experience on the Great Kanawha led to the invention of what should properly be called the Scott hurter, which is now exclusively introduced into the later dams of the Kanawha. It is fully described in Mr. Scott's paper, but he is too modest to claim all the credit which is his just due. It gives me great pleasure to award it to him.

On another occasion I used the language below concerning Mr. Scott, which is entirely true:

"I found him on the Kanawha when I took charge of it. He has been either the Principal Assistant Engineer on the Kanawha, or the Resident Engineer, almost, if not altogether, continuously since that time, I think, and has had more to do with the superintendence of the construction and operation of the movable dams on the Kanawha than any other person, and my opinion is that his experience, in comparing the Pasqueau hurter with the tripping bar, is larger than that of any other man in the world."

Under the supervision of Mr. Scott several modifications of the details of the manœuvres of the dams have been made, and all in the line of improvement.

J. P. Frizell, M. Am. Soc. C. E. (by letter).—The movable dam is one of the most remarkable developments of modern hydraulic engineering. There are probably few engineers of any experience in river work who have not, at times, been impressed with the great advantage that would result from the ability to set up a dam in low stages of the river, and leave a free passage for the water in times of flood. The introduction of this form of construction in the United States has been attended with great opposition, an opposition founded for the most part upon misapprehension. The idea of maintaining a 10, 12 or 15 ft. head of water by a series of light wooden rickets resting against props appears ridiculous to people who are accustomed to regard a solid mass of masonry, as wide as it is high, as none too strong for that purpose. They fail to consider that the forces which act distinctively upon a dam are created by the dam itself. A mass of water is formidable only when set in violent motion. A movable dam is largely free from the dangers to which fixed dams are exposed. It exists only when

the flow of water is slight. It controls the stream only in its times of tameness and docility. It lies safe and harmless on the bed of the stream when the latter is wild and violent.

These remarks apply more especially when the movable dam extends entirely across the stream. When it simply controls a pass in a fixed dam it is liable to occasion a scour more dangerous than if the entire dam were fixed.

The chief advantage to navigation contemplated by the use of the movable dam is that it subjects vessels to the delay of passing the lock only during the lowest stages of the stream, leaving a free channel when the flow of the stream is sufficient for the requirements of navigation without the aid of slack water. Also that on streams liable to great freshets it admits of a cheaper construction in not requiring lock walls and the abutments of the dam to go above high water.

Four general types of the movable dam have been used to a greater or less extent in works of navigation. The first to be mentioned is the bear trap gate, of which all engineers have a general idea. This consists of two leaves extending entirely across the channel, turning on horizontal parallel hinges, attached to the floor of the channel. The taller of the two leaves is on the up-stream side, and, when the channel is open, lies folded over the shorter. Water from a higher source, admitted to the chamber under the gates, causes them to rise, and, being withdrawn, causes them to fall. The top of the shorter leaf remains in contact with the side of the longer one, and they form in connection with the side walls and bottom of the channel a closed space sufficiently water tight for the purpose. The bear trap gate is an American invention, and was first applied by Josiah White, of Pennsylvania in 1818, as a lock gate on the Lehigh Canal. It has since found many applications as a movable dam, properly so called, on the rivers of Pennsylvania. In May, 1887,* a board of engineer officers reported that dams on this principle were perfectly practicable for openings of 60 ft. and a lift of 12½ ft. This construction was applied to dam No. 1 on the Monongahela River,† about 1880, to an opening of 120 ft. The difficulty here encountered was that the dam did not rise uniformly, one end rising considerably in advance of the other.

This device exceeds every other form of movable dam in the ease with which it is manipulated. It has no disadvantages that could not be overcome by perseverance and intelligence. It is to be regretted that it has not been more extensively applied. The construction of such a dam was commenced several years ago at Beattyville, on the Kentucky River, but was abandoned for reasons wholly unconnected with its merits.

The second system is the Thenard shutters, first applied by

* Report of Chief of Engineers, U. S. A., 1887, Vol. 3, p. 1882.

† Report of Chief of Engineers, U. S. A., 1885, Vol. 3, p. 1862.

Thenard on the River Isle, in France, in 1828. This consists of two parallel sets of shutters extending along the top of a low masonry dam to which they are hinged. When the channel is open, they lie extended on the masonry, each set falling from the other. The up-stream set is confined by latches releasable by a tripping bar. When the dam is to be raised, the attendant releases the up-stream set and the current raises them into an erect position. This stops the whole flow of the stream and holds it in the pool above the dam, leaving the masonry accessible, so that the down-stream set of shutters can be erected and confined by props. When the water rises to the top of the up-stream set, and the pressure comes on the down-stream set, the former fall and are automatically latched. When the channel is to be opened, the down-stream shutters are dropped by a tripping bar.

This system appears to have worked admirably in the cases to which it has been applied. In the application described it did not serve as a dam, but as what we term flash boards. It is manifest that, applied to a navigable channel, the lower shutters would not become accessible when the upper ones are raised; but with suitable modifications, the principle is perfectly applicable to that case.

The third system, and one which has found extended application on European rivers, is the Poiree or needle dam. In this the framework of the dam consists of a series of trestles, which, at the same time, sustain a bridge for the convenience of workmen. The water-tight diaphragm consists of planks or joists called "needles," which are brought on to the bridge from a storehouse on shore, and are inserted from above. When the channel is open, the trestles lie flat on the floor, their feet resting in bearings whose axes extend up and down stream. They are raised by chains worked from the wall or abutment. The objections to this system are mainly the difficulty of maintaining a sufficient degree of tightness, and the large amount of material to be handled at each manœuvre of the dam.

The fourth system is the Chanoine, which forms the subject of Mr. Scott's paper. In this, each trestle carries the corresponding part of the water-tight diaphragm of the dam, called a wicket. The feet of the trestle rest in bearings whose axes are transverse to the current and in line with the dam. The top of each trestle has two journals to which the wicket is articulated, and a third to which is jointed the prop for supporting the trestle. When down, the trestle and its props lie extended in the same line on the floor, and the wicket lies over them. A strong pull on the up-stream end of the wicket raises the trestle together with the wicket, the prop following till it reaches a notch or step in the floor, when the whole articulated system rests firmly, the wicket hanging suspended near its middle part. Depressing the up-stream end of the wicket, it rests on a suitably formed sill, the wicket standing in a nearly vertical position and forming a part of

the dam. A bridge formed by trestles similar to those of the Poiree dam, though lighter, is usually set up for convenience in erecting the Chanoine dam, though the work may be done from a boat held by cables. The wickets, of course, cannot touch one another or approach within less than from 1 to 2 ins. These openings are most conveniently closed by square scantlings which rest in them with the diagonal vertical. These scantlings are the only extraneous parts of the dam. The trestle is thrown down by releasing the foot of the prop.

This brings us to the principal point of Mr. Scott's paper, which deals specially with the methods of accomplishing this result. Two methods are in use for that purpose. The first consists of a tripping bar, which is a stout iron rod reaching the whole length of the movable dam, and susceptible of a longitudinal movement by means of gearing at the lock or abutment. Projections, or studs, on this rod successively engage the foot of each prop and draw it off its seat. These studs are spaced a little wider than the wickets, so that the tripping bar engages with but one prop at a time, though, of course, the studs can be so spaced as to trip any number of wickets at once. The common arrangement is to throw down singly till the pressure is sufficiently reduced, then in two's, then in three's, etc. This, I understand, is the method not found satisfactory on the Kanawha. The second method consists in a peculiar construction of the plate on which the foot of the prop slides, in raising and lowering the trestle, which plate has retained its French name of hurter. It is so formed that the foot of the prop pursues one path while the trestle is rising, and another while it is descending. In the first path there is a notch or step into which the foot of the prop falls and rests firmly when the trestle has reached its proper position. When the trestle is drawn further forward, the foot of the prop follows, and reaches a position when its path terminates. It slides sidewise into the second path which offers no obstruction to its backward movement, and allows the trestle and wicket to fall freely. When the trestle is down, the foot of the prop finds itself again in the path which it follows while rising.

This style of movable dam is the one that has found greatest acceptance in this country. Whether it is the best that can be devised is a question of minor importance. That it effectually and practically fulfills the requirements of a movable dam no longer admits of doubt. It is not inordinately expensive, and its cost may be expected to diminish with extended use and experience.

It appears to me that a great error was committed by the United States Engineer Department in not adopting movable dams for the Cumberland River. The stream is navigable some six months in the year as far as Smith's Shoals, some 350 miles above Nashville, in Tennessee. For a long distance above Nashville its slope is 8 ins. to the mile, and still less below that place. It flows in a cañon worn out

of the rock 100 ft. or more in depth, half filled with alluvial deposit. It is liable to extreme floods of over 50 ft., and rises of 35 ft. are not rare. To raise lock-walls and abutments above high water is out of the question, not only on account of the expense, but on account of the contraction of the flood water-way. The locks must often be wholly submerged and out of use. With permanent dams there cannot fail to be times when boats can neither pass the lock nor dam. Like all the smaller western rivers, this stream is frequented by stern-wheel steamers. A system of slack-water navigation is contemplated for this stream, involving some 30 locks and dams of 10 or 12-ft. lift. A rise of 35 or 40 ft. would, no doubt, make nearly smooth water over a 10 or 12-ft. dam. But with a rise of 20 or 30 ft., which would put the lock out of use, there would be a fall at the dam such that a stern-wheel steamer going up would on reaching the upper level find its wheel out of the water, helplessly beating the air. No river in America presents more distinctly the conditions calling for movable dams.

Three boards of engineer officers have considered this question. The first recommended fixed dams; the second advised movable dams; the third reversed the latter decision and redecided for fixed dams. No dams, however, will be required till several of the locks are completed, and there will be abundant time to revise the latter decision.

Almost every dam built for industrial uses calls for some application of the principles of movable dams. Most such dams are liable to frequent floods, giving a depth of 6 ft. or more on the dam. This height affects the lands bordering the streams as far up as the influence of the dam extends. The damages incident to such a rise of the water have to be paid in obtaining the right to build the dam. These damages are but very slightly aggravated by holding the water permanently at the ordinary flood level, to the great advantage of the water-power in respect to fall and pondage. The permanent part of the dam cannot be built any higher, as that would increase the flood height. The conditions call for some addition to the height of the dam, which can be erected in low water and dropped on the approach of a flood. The arrangement of flash boards now in use is a very crude expedient, very imperfectly meeting the requirements of the case. It consists of loose boards held by the pressure of the water against pins inserted in holes drilled or bored in the top of the dam. This arrangement is limited in height, never exceeding 4 ft. The obstruction is not removed in time of flood, except in so far as the pins are broken or bent and the boards carried away at such times. It has often appeared to me that, in a dam constructed with reference to this requirement, this exceedingly primitive and imperfect device might very advantageously be replaced by something like the Thenard shutters above referred to. More especially are such arrangements applicable on southern rivers, where ice, that implacable enemy of all refinements in river construction, does not exist.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

709.

(Vol. XXXI.—May, 1894.)

STORAGE AND PONDAGE OF WATER.—DISCUSSION ON PAPER No. 688.*

By WILLIAM E. WORTHEN, A. FTELEY and CHARLES B. BRUSH.

WILLIAM E. WORTHEN, Past President Am. Soc. C. E.—The facts collected by the author of the paper are right, but to undertake to form a general rule of yearly averages of water flows I object to, unless nothing better can be got. To an expert in water suits the best evidence of the capacity of a stream is from the daily flows for as many years as possible, plotted in an extended profile, and, on this, continuous horizontal lines of different flows per day, from which can be determined how much can be obtained daily through the whole profile, what pondage will be necessary, reference being had to what can be readily secured, discarding excessive flows which raise the average of flows so much, and cannot be ponded. On the lower Bronx there are three months in the year in which there is little, if any, power to be obtained without pondage, and the necessary land for pondage can not be got.

There is another matter not referred to in the paper, but pertinent; that is, the expert testimony in the cases of diversion of water, which is almost as conflicting as medical testimony. My aim is to be consistent

* "Storage and Pondage of Water," by Joseph P. Frizell, M. Am. Soc. C. E. Vol. XXXI, p. 29.

on whichever side I am retained, and inform my client of my view of his case, and especially of the points on which I cannot support him; but expert testimony should be for the court, and not for clients.

Mr. FTELEY.—I recollect an instance which illustrates the remark just made, that expert testimony should be for the court and not for the clients.

The case on hand was of some importance, and experts were engaged on both sides; but, although their testimony could not be objected to as regards fairness, the facts before the court were so presented that important points were omitted, thus favoring an appearance of contradiction between the experts on each side. From conversations subsequently held between the experts, with all the facts at hand, I do not doubt that, if consulted by the court, with all the evidence before them, they would have substantially agreed.

CHARLES B. BRUSH, M. Am. Soc. C. E.—It seems to me that Mr. Fteley has pointed out the cause of a great deal of the difference that appears in expert testimony. When experts are asked to give their opinion on a subject, and are asked in a general way, so that they are free to give that opinion, that is one thing; but as a rule that is not the way they are asked their opinion in causes of this kind. Almost invariably there is an assumption made, and, based on that, they are asked what results follow, and they are confined by the very nature of the question that is asked them.

Mr. WORTHEN.—What effect has the cross-examination?

Mr. BRUSH.—The cross-examination does not always bring out the facts on the other side of the case. It is true, that if an expert should really be given the opportunity of saying frankly what he believes on any subject, a great deal of this difference would disappear; but the expert is confined to certain lines of examination, so that the cross-examination does not always obviate this difficulty. He gives part, but, as in the case that Mr. Fteley brought up, the value given by the experts on the basis of an assumed condition was a certain amount. On the other side, on another assumption, a different result was reached. Afterwards, when the experts came together, there was no difficulty in coming to an agreement, taking all the facts as they found them.

It is most unfortunate that experts are confined to certain lines to which the lawyers on either side feel that the interests of their case requires that they should be confined.

Mr. FTELEY.—I wish to ask Mr. Worthen a question.

If you are retained by one party, and, notwithstanding your desire to give true testimony, even if it is to injure your client, it may happen that you know of weak points, unknown to the other party, and which, if brought up, would injure your case, would you then consider, under your oath to "tell the truth, all the truth and nothing but the truth," that you are in duty bound to disclose such weak points?

Mr. WORTHEN.—I say, no. I do not consider that it is my duty to offer evidence of the weakness of my case at all; but I state distinctly to the lawyers on the side on which I am retained, what the position is.

In a case at Trenton, for the removal of the dam across the Delaware, the answers which I gave in court to the questions on direct examination left a wrong impression, but it was relieved by the question of the judge. And in most cases, I have found that the cross-examination or interrogatories of judge or of commissioners bring out the truth; at any rate, I depend on that. The phrase "to tell the truth, the whole truth and nothing but the truth" is construed by the court and counsel to mean truthful answers to the questions asked. It is, probably, within the knowledge of every one who has been in any court, that where a witness offers anything explanatory, for the attorneys to say that he was not asked that, and for the judge to say to him he had better confine himself to the question.

I try to make answers full and explanatory of the rules of flowing water, and decline to appear as witness where I must be considered as a retained counsel, to give one-sided evidence under oath.